STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 40

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

REPRINTS AND PRELIMINARY REPORTS

Supplement to the "Proceedings" and "Meddelanden" of the Institute

Stability and Strengthening of Rock Tunnels in Scandinavia

1. **Correlation of Seismic Refraction Velocities and Rock Support Requirements in Swedish Tunnels**

   Owen S. Cecil

2. **Problems with Swelling Clays in Norwegian Underground Constructions in Hard-Rocks**

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STOCKHOLM 1971

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PREFACE

The Swedish Geotechnical Institute herewith presents a publication on rock mechanics, which deals with practical problems. The publication consists of two papers. The first is based on results from measurements in tunnels in the northern Sweden and correlates the support requirements in rock tunnels with the longitudinal seismic refraction velocity. The second paper is a report on experiences gained from investigations of swelling clays in rock tunnels in Norway.

The author of the first paper, Mr Owen S. Cecil from the University of Illinois, USA, worked with the subject during 1966-68 at the Swedish Geotechnical Institute. The investigation was supported by grants from the Swedish Power Board and the Swedish Fortifications Administration and done in cooperation with the Rock Mechanics Committee of the Swedish Academy of Engineering Sciences (IVA).

After his return to USA, Mr Cecil supplemented his work with, e.g., laboratory model studies which resulted in a Ph. D. thesis at the University of Illinois. The Institute plans to issue the thesis in its Proceedings series.

The second author is Professor Rolf Selmer-Olsen of the Technical University of Norway, Trondheim. His work is based on a lecture given to the Swedish Geotechnical Society in the beginning of 1970. The Institute wants to thank Professor Selmer-Olsen and the Society for making it possible to publish his valuable contribution.

Stockholm, September 1970
SWEDISH GEOTECHNICAL INSTITUTE

CORRELATION OF SEISMIC REFRACTION VELOCITIES AND ROCK SUPPORT REQUIREMENTS IN SWEDISH TUNNELS

Owen S. Cecil

SWEDISH GEOTECHNICAL INSTITUTE

CONTENTS

			Page
Abstract			1
1.	INTRODUCTION		3
2.	INVESTIGATIONS IN RÄTAN TAILRACE TUNNEL		5
	2.1	Measurement Technique	5
	2.2	Presentation of Results	7
	2.3	Discussion of Velocity Data	8
	2.31	General	8
	2.32	Intact Rock Seismic Velocities	17
	2.33	Analysis of Velocity Data from Tunnel	18
	2.34	Seismic Velocity-Tunnel Support Relationship	27
	2.35	Application of Results to Surface Velocity Data	30
	2.36	Comparison of Velocity Data with Support Quantities	33
	2.4	Discussion of Supplementary Data	37
	2.41	Bedrock Surface Profiles	37
	2.42	Destressed Rock at Tunnel Periphery	38
3.	APPLICATION OF RÄTAN RESULTS		40
	3.1	Seitevare	40
	3.2	Bergvattnet	48
4.	CONCLUSIONS		48
	Acknowledgements		51
	Appendix. Geologic Descriptions of Tunnel Units, Rätan		53
	References		57

ABSTRACT

Seismic refraction measurements in a Swedish tailrace tunnel have indicated a crude relationship between the longitudinal seismic velocity and the approximate degree of support necessary for loosening instability in medium to large tunnels (greater than 30 sq. meters in cross-sectional area) in hard crystalline rocks. The seismic velocity ratio, defined as the ratio of the longitudinal velocity for a particular rock condition to the longitudinal velocity for intact rock of the same type, is related to support requirements in the following way:

Seismic velocity ratio	Support requirement
>0.90	none to spot rock bolting
0.80 - 0.90	light shotcrete (<5 cm) frequently in combination with rock bolting
<0.80	rock bolting, heavy shotcrete (>5 cm, two or more applications)

Exceptions to the classification do exist, particularly for large blocky ground that requires pattern rock bolting, and for cases where a single rock mass discontinuity causes difficulty in an otherwise stable ground condition. The limit values of the seismic velocity ratio for different support requirements are not intended to be absolute limits, but only crude guidelines. A more general designation for rock quality and support requirements is to consider rock with a seismic velocity ratio of less than 0.8 as a weak zone that requires some form of support, usually at least 5 cm of shotcrete.

This concept has been applied to surface seismic refraction investigations at three tailrace tunnels in northern Sweden. The predicted support requirements compare reasonably well with those conditions actually encountered in the tunnels, and it is concluded that for shallow tunnels (rock cover less than 100 meters) in hard crystalline rocks, seismic refraction velocities generally give a better indication of potentially poor rock conditions than do randomly located drill holes.

1. INTRODUCTION

The usefulness of seismic refraction measurements for the planning of underground construction in rock has been known in Sweden for at least twentyfive years. The earliest applications were primarily for the determination of the depth to bedrock, or the thickness of soil cover (Hasselström, 1951). Refinements in the interpretation technique led to further application of the refraction method for the accurate determination of bedrock velocities. Since 1959 the method has been used successfully in Sweden for the localization of weak zones at the bedrock surface (Scherman, 1959, Hasselström et al., 1964).

More recently the seismic refraction technique has been used in the pilot bore of the Straight Creek highway tunnel in the Rocky Mountains, Colorado, USA, to establish relationships between rock quality and various construction parameters, such as rates of advance and spacings of steel sets (Scott et al., 1968). The results from these studies show that the longitudinal seismic velocity as determined from the seismic refraction technique provides a reliable numerical index of rock quality that can be related to the economic and engineering aspects of tunneling. It was concluded from the Straight Creek studies that, if such measurements should prove to be valid for indexing rock quality for a wide variety of geologic conditions, then it would be worthwhile to direct efforts towards the development of methods to measure seismic velocities in "feeler" holes that could be driven out ahead of the working face during the driving of the tunnel. This method of investigation might prove to be particularly suited to mechanical "mole" tunneling.

A potential use in Sweden of such seismic velocity-tunneling relationships as those developed in the Straight Creek tunnel also lies in their application to velocities determined by conventional ground surface refraction measurements. In order to test the applicability of seismic velocity data as a measure of rock quality for Swedish bedrock and tunneling conditions, an investigation was made in the tailrace tunnel at the Rätan hydroelectric project in Jämtland, northern Sweden. Ground surface seismic refraction measurements over the entire tunnel line had been made during the preliminary investigation stage at Rätan. These investigations were complemented with a total length of 1010 meters of seismic refraction profile that was shot in the tunnel during the construction period. The purpose of these latter measurements was the following:

1. To indicate the most reasonable interpretation of the velocities provided by surface refraction measurements.
2. To determine the significance of seismic velocity as a measure of rock quality with respect to different tunnel support requirements in loosening ground conditions.
3. To investigate the extent of the destressed zone at the periphery of the tunnel.

The first of these items is of primary interest for the preliminary stage of tunnel planning, while the second two are of more concern for the question of rock support requirements during construction. The extreme variation of conditions of the several different rock types in the Rätan tunnel presented an ideal opportunity to test the applicability of seismic velocities to the three items listed.

The results of the tunnel measurements at Rätan led to a simple empirical criterion for the prediction of bedrock weak zones. This criterion is applied to the preliminary investigations for the Seitevare hydroelectric project in northern Lappland. Observations of rock conditions in the completed tunnel at Seitevare were made and the comparisons of seismic velocity and support measures are discussed. Correlations between seismically determined weak zones and tunneling conditions are also presented for a short section of tunnel at the Bergvattnet hydroelectric project in northern Sweden.

2. INVESTIGATIONS IN RÄTAN TAILRACE TUNNEL

2.1 Measurement Technique

The tunnel measurements at Rätan were carried out by Craelius Terratest AB of Stockholm (formerly AB Elektrisk Malmletning). The measurements were made in essentially the same manner as the surface refraction measurements made by Craelius during the preliminary investigations. Descriptions of the principle of seismic refraction measurement are available elsewhere and will not be given here (Hasselström, 1951, Scherman, 1959). Some of the technical details of the Rätan tunnel measurements are given in the following paragraphs.

A geophone spacing of 5 meters was used for most of the measurements in the tunnel. Shorter sections of profile were shot with 2.5- and 1-meter spacings to investigate in more detail the extent of the destressed zone at the tunnel periphery.

The geophones were of the ordinary spring suspension, moving coil type (Hall-Sears manufacture) with frequency response in the 5-300 cps range. Recording was done photographically at paper speeds of 2.5-5 meters/second. Arrival times could be determined with an accuracy of about ± 0.3 millisecond.

The set up for shotpoints and geophones is shown in Fig. 1. Shot

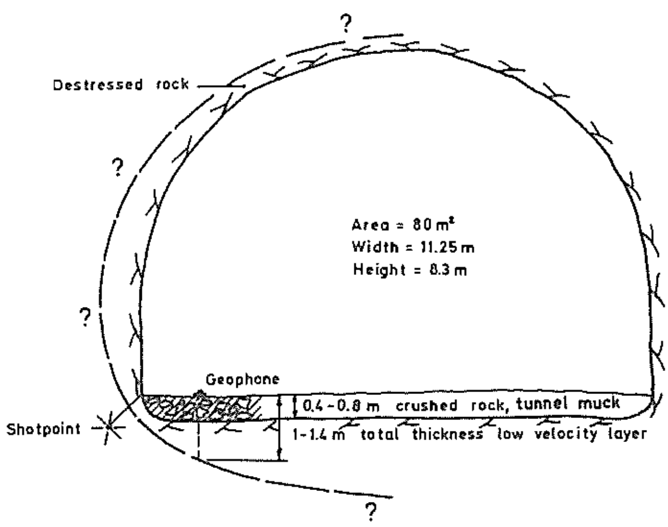

Fig. 1 Geophone setup for tunnel measurements.

holes were drilled one meter deep into the corner of the tunnel wall and floor. A 1- to 2-inch stub of dynamite was used for the shot charge. The geophones were originally screwed into 18-inch-long steel dowels that were firmly implanted in the tunnel floor, but it was later found easier and equally effective to screw the geophones into 4-inch-diameter, 1-inch-thick steel plates and place the latter directly on the tunnel floor, as shown in Fig. 2. A photograph of the detonating and recording set up is shown in Fig. 3.

Fig. 2 Geophone-steel plate pickup for tunnel measurements.

Hall-Sears seismometer, spring suspension, moving coil type, frequency response range 5-300 cps; pickup spacing 1-5 m.

Fig. 3 Detonating and recording equipment for tunnel seismic refraction measurements.

A normal shot line consisted of 20 geophones on 5-meter spacings. Shot points were located at both end points, at 20 meter intervals within the geophone spread, and at 25 and 50 meters from each end of the spread.

Clean records were obtained for most of the entire profile length of 1010 meters. The only disturbances were experienced in a diabase and amphibolite inclusion that apparently conducted leak currents from the tower of a 20, 000 volt high tension line that crosses the tunnel at the ground surface above that station of the tunnel. It was generally not possible to detect shear wave arrivals from the records.

2.2 Presentation of Results

The results of the tunnel seismic measurements, together with the surface seismic profiles, are shown in Figs. 4-7. Fig. 4 is a general orientation map of all the preliminary investigations along the tunnel. Profiles 301, 302, and 303 are the tunnel profiles. The criterion used to delineate low velocity zones will be discussed in the next section. Results from magnetic measurements that were made during the driving of the tunnel are also shown in Fig. 4 (Larsson & Stanfors, 1966). It is seen that the high values of magnetic susceptibility correspond approximately to the larger zones of diabase and amphibolite in the tunnel.

Figs. 5-7 give a more detailed picture of the refraction data, together with a schematic map of the rock conditions in the tunnel and the primary support work that was done during the driving of the tunnel. In most cases the primary support also served as permanent support. All of the stability difficulties encountered at Rätan involved loosening and subsequent fall of rock material from the tunnel periphery. None of the instability was caused by high stresses in the bedrock.

The detailed geologic mapping of the tunnel that is used in this report was done by Mr Magne Bekkelund of Sydsvenska Kraft AB.

2.3 Discussion of Velocity Data

2.31 General

The seismic refraction profiles in the tunnel were shot by the same company and in the same manner as those on the ground surface. Similarly, the arrival time data has been interpreted in the same manner for all of the profiles discussed in this report. There are therefore no differences in any of the data that can be attributed to differences in measurement or calculation technique. The seismic velocities shown for the surface profiles are not those given in the original reports, but rather the reinterpreted values that have been issued by Craelius Terratest in a supplementary report (AB Elektrisk Malmletning, 1963).

The velocities shown in plan on Figs. 5-7 are those of the respective profiles reported by Craelius. The location and extent of the velocity zones directly over the tunnel from section 0-1000 downstream have

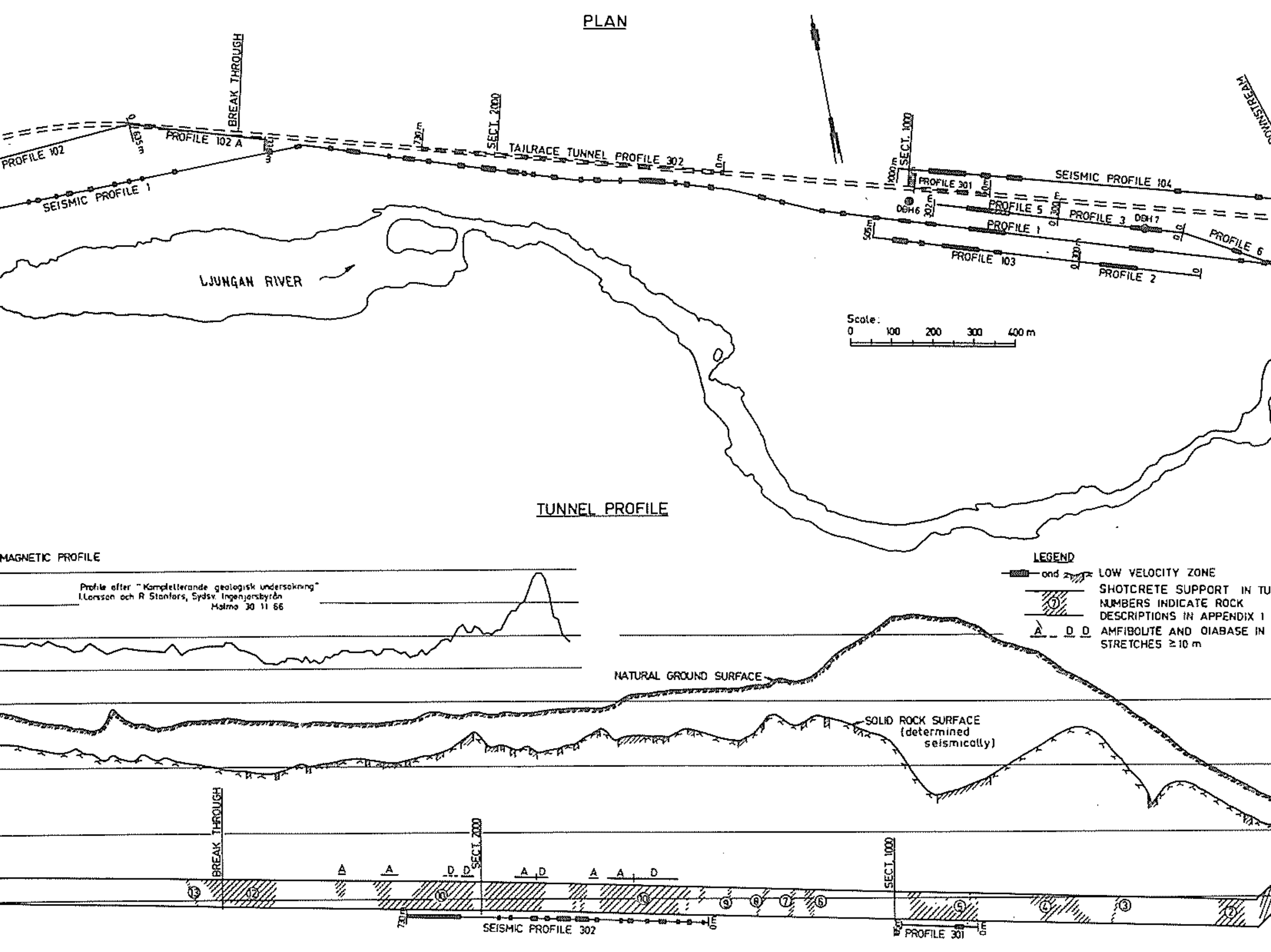

Fig. 4 Rätan power station, tailrace tunnel. General plan of rock investigations and tunnel reinforcements.

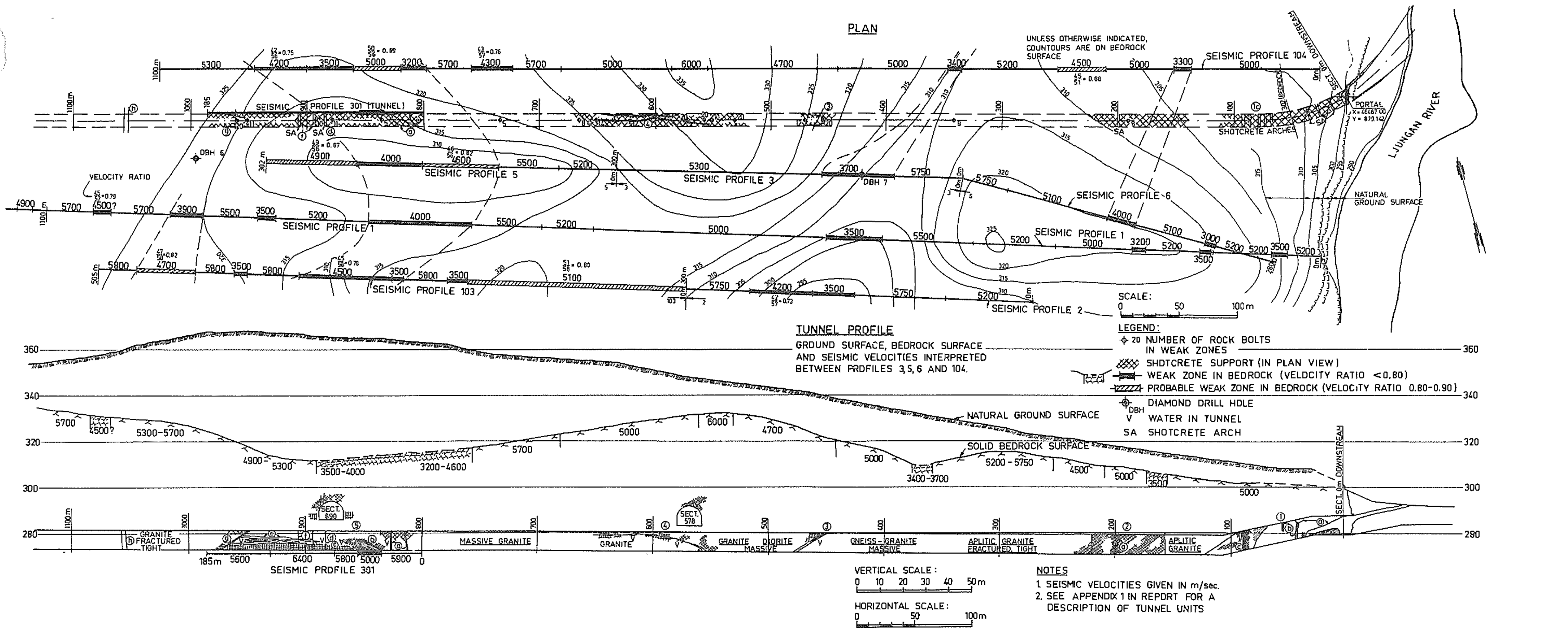

Fig. 5 Rätan power station, tailrace tunnel. Seismic data and geologic mapping, section 0 to 1100.

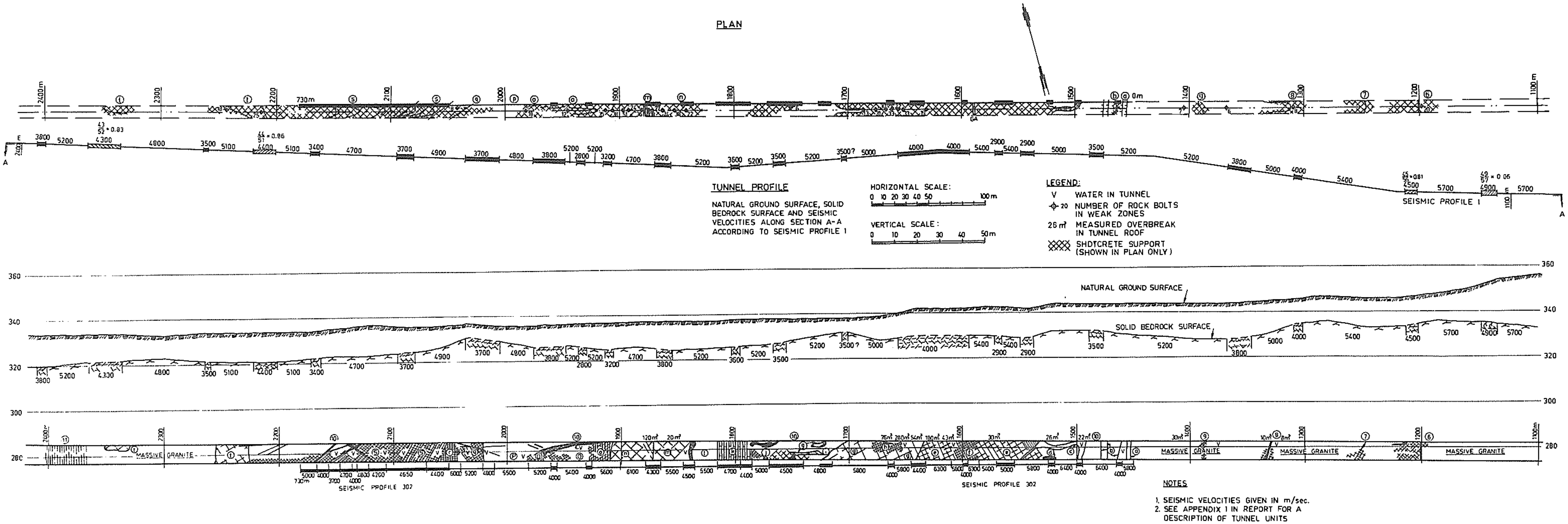

Fig. 6 Rätan power station, tailrace tunnel. Seismic data and geologic mapping, section 1100 to 2400.

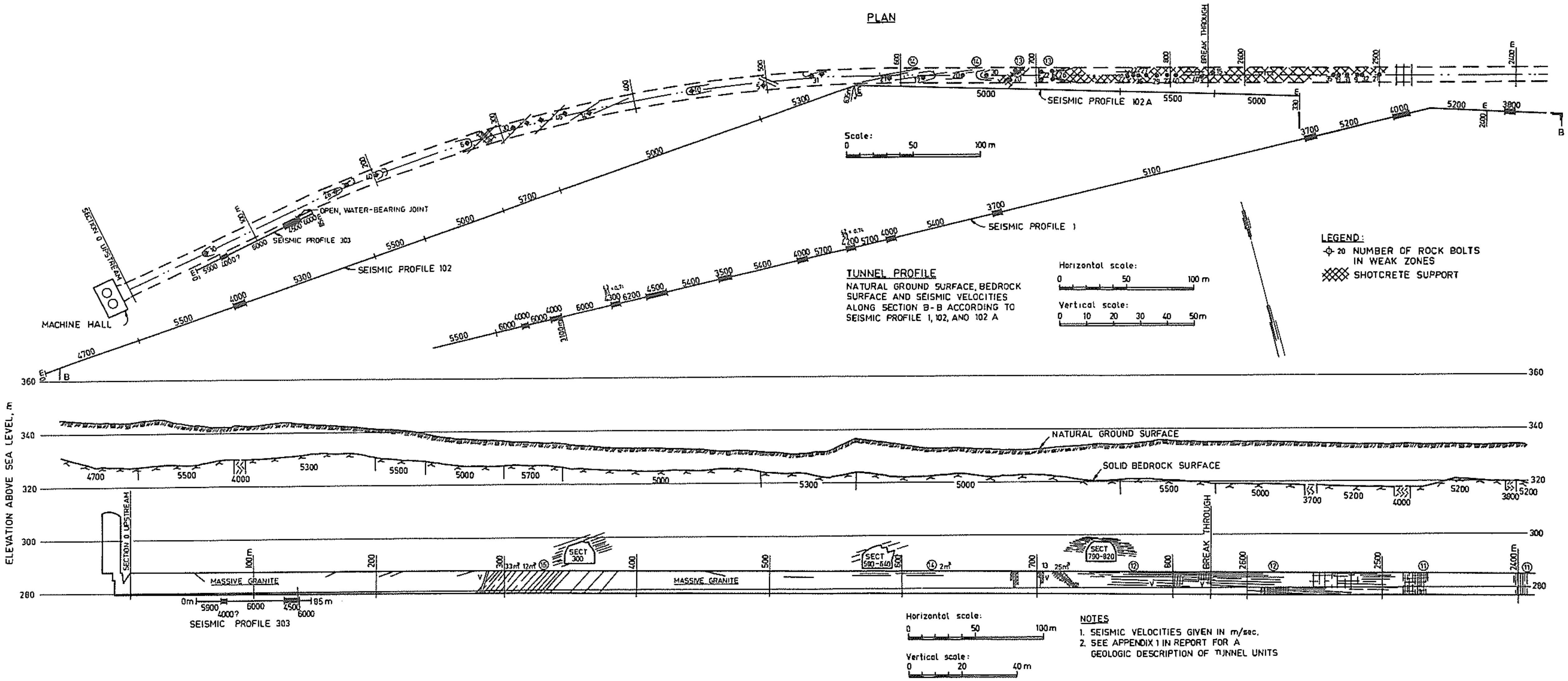

Fig. 7 Rätan power station, tailrace tunnel. Seismic data and geologic mapping, section 2400 downstream to section 0 upstream.

been interpreted between parallel profiles by the writer. These values for this section of the tunnel are the ones shown in profile on the figures. The velocities over the remaining portions of the tunnel, that is upstream from section 1000, have been taken directly from the nearest profile.

2.32 Intact Rock Seismic Velocities

The tunnel refraction profiles at Rätan included measurements in the following rock types: coarse-grained granite; fine-grained aplitic granite; amphibolite; and diabase. Because of the very detailed round-by-round geologic mapping that was done in the tunnel, it has been possible to correlate the seismic velocities with rock types, support quantities, and in a few cases, Schmidt N-hammer hardness values.

The four different rock types, the maximum seismic velocities in these different rock types, and Schmidt N-hammer hardness values are given in Table 1.

TABLE 1 Maximum Seismic Velocities and Schmidt N-hammer Rebound Hardnesses for Rätan Rock Types

Rock Type		Velocity of High Quality Rock m/sec	(ft/sec)	Schmidt N-hammer Hardness
Coarse-grained Granite	a	5500	(18,000)	---
	b	6000	(19,700)	55
	c	6400	(21,000)	---
Fine-grained Aplitic Granite		6400	(21,000)	67
Amphibolite		6100	(20,000)	60
Diabase		6300	(20,700)	63

After the seismic measurements had been completed and analyzed, it became evident that three different hardnesses of coarse-grained granite exist along the profiles. The 6000 m/sec rock corresponds to the granite in which the Schmidt N-hammer hardness of 55 was obtained. Both the velocity and the hardness values are the maximum values for the corresponding rock types. The rock conditions to which these properties correspond are those that do not require any support to maintain the desired tunnel shape. The rock is jointed in some cases, but tight. The approximate relationship between seismic velocity and Schmidt N-hammer hardness is shown in Fig. 8.

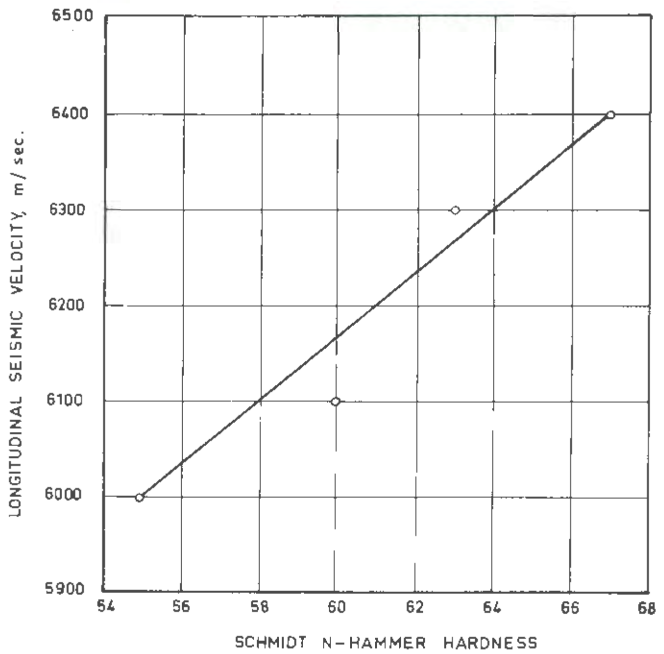

Fig. 8 Relationship between Schmidt N-hammer rebound hardness and longitudinal seismic velocity.

2.33 Analysis of Velocity Data from Tunnel

The most striking feature of the data from the tunnel profiles is that the seismic velocities are considerably higher than those from the surface measurements. This phenomenon can be attributed to two factors:

1. An overall loosening of rock at the surface that has occurred after glacial retreat. Weak zones at the bedrock surface are probably considerably more relieved and loosened than sounder rock several meters below the rock surface.

2. A general tightening or increase of confining stress with depth that effectively holds jointed rock together.

A comparison of velocities from the tunnel and from the bedrock surface for different sections along the tunnel is given in Table 2.

The fourth column in the table is the ratio of the tunnel velocities to the surface velocities. For the high velocity or high quality rocks that require no support this ratio varies from 1.11 to 1.17, indicating that the tunnel velocities are from 11 to 17 per cent higher than those near the bedrock surface. The difference is from 15 to 38 per cent for low quality rock in weak zones that require support. These differences suggest that a direct comparison of the velocity values from the tunnel to those from the surface profiles is not possible. In the next section a more detailed study of the tunnel data is made to arrive at a general concept for the treatment of surface refraction velocities.

The refraction measurements in the tunnel have provided the opportunity to compare different magnitudes of velocities with known rock conditions and support requirements. Extensive field observations, together with the geologic mapping have provided a good insight into the geology and stability conditions along the entire tunnel.

In the usual application of seismic velocities to the assessment of bedrock conditions in Sweden it is assumed that velocities equal to or less than 4000 m/sec are representative of poor quality bedrock. After preliminary studies of seismic reports and rock conditions in tunnels at several sites in northern Sweden, the writer concluded that such a simple interpretation is correct, but that a more inclusive interpretation could be made of velocity data if one first had a suitable framework or reference system for velocities in different rock conditions. The results from the tunnel measurements in Rätan provide a basis for such a reference system. The profiles in the tunnel were planned so as to include a wide variety of rock types and structural conditions. Profiles 301 and 302 were placed along the worst rock in the tunnel, whereas Profile 303 was placed in very sound granite to obtain a control velocity for the highest quality rock in the tunnel.

The velocity range along all of the tunnel profiles of from 3700 m/sec

TABLE 2 Comparison of the Relative Magnitudes of Tunnel Velocities and Surface Velocities

Geologic Unit	(1) Tunnel Velocity m/sec	(2) Surface Velocity m/sec	Ratio (1)/(2)
<u>High Quality Rock</u>			
Granite, Sect 100, Upstream	6000	5300-5500 <u>5400</u>	1.11
Granite, Zone 5	6400	5500-5700 <u>5600</u>	1.14
Diabase, Zone 10c	6300	5400	1.17
<u>Low Quality Rock</u>			
Granite, Zone 5	5000	3500-4000 <u>3800</u>	1.32
Diabase, Zone 10e	4000	2900	1.38
Fine-grained Granite, Zone 10s	4000-4650 <u>4300</u>	3700-3800 <u>3750</u>	1.15
Fine-grained Granite, Zone 10c	4000	3500	1.15

Note: The ratios (1)/(2) have been computed from the average (underscored) velocities.

to 6400 m/sec is alone an indication of the variation in rock quality in the tunnel. However, to make any valid comparisons of the velocities in different sections of the tunnel for different support conditions, it is first necessary to normalize all the velocities so that differences due to intact rock properties are eliminated.

The tunnel seismic velocity for a given rock condition in a given rock type has been normalized by dividing by the maximum seismic velocity (as given in Table 1) for that particular rock type. The ratio so obtained has been termed the seismic velocity ration (SVR). This idea of establishing a "base velocity" for each rock type is similar to that used by Lakshmanan (1966). Deere's velocity ratio is a similar tool for normalizing the field seismic velocity (Deere et al., 1967). Onodera (1963) was apparently the first investigator to propose such a quality index for in situ rock.

The seismic velocity ratio is a normalized indicator of rock quality, that is, a combined measure of the following items:

1. Degree of jointing,
2. Degree of alteration,
3. Degree of openness of joints, or looseness of rock structure.

The seismic velocities for the unsupported rock conditions along the tunnel refraction profiles are shown in Table 3. The conditions are grouped according to the different rock types and maximum velocities. Where it has been possible to project weak zones in the tunnel to low velocity bedrock surface zones, the bedrock surface velocities are shown in parentheses, together with the adjacent high velocity rock at the bedrock surface. The tunnel seismic velocity ratios are computed as described previously. The bedrock surface seismic velocity ratios are computed from the velocity values shown in parenthesis. It is assumed that the rock immediately adjacent to low velocity zones is the same type as that in the low velocity zone. The support requirements for the low quality rocks have been broadly classified as follows:

CLASS 1 no support or light spot rock bolting

CLASS 2 light shotcrete (< 5 cm), frequently in combination with rock bolting

CLASS 3 heavy shotcrete (> 5 cm, two or more applications)

The distinction in class of required support is not always sharp. Some typical rock conditions are shown in Figs. 9-11.

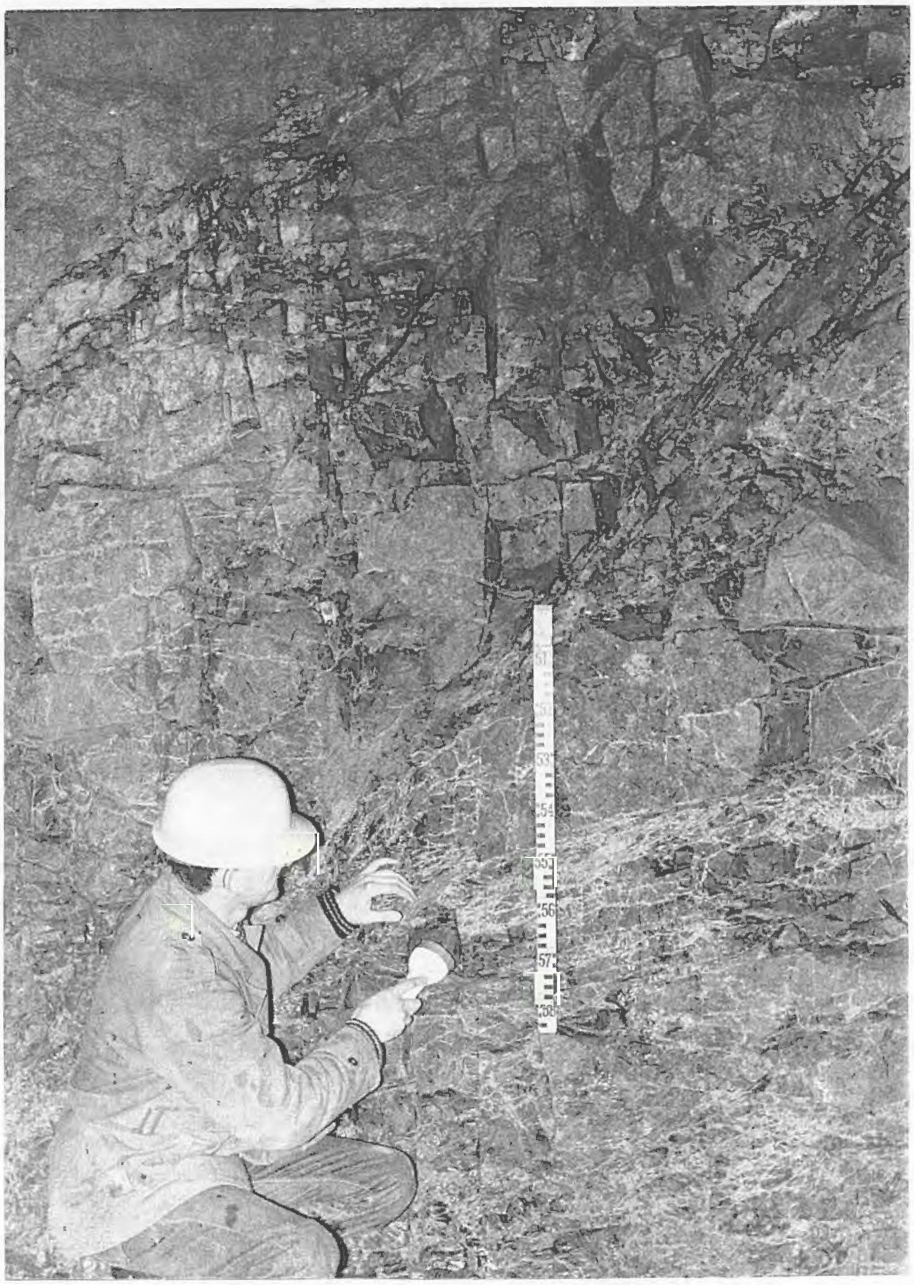

Fig. 8 Rätan tailrace tunnel, section 1520.

Tunnel Unit 10d; shear zone in aplitic granite; crushed rock along diagonal seam is partly altered; some joints are filled with a white rock flour material; joint spacing less than one cm to 20 cm; seismic velocity 4000 m/sec; seismic velocity ratio 0.63; support class 3.

Fig. 10 Rätan tailrace tunnel, section 2000.

Tunnel Unit 10p; coarse-grained granite; joint spacing 5 cm to 60 cm; seismic velocity 5500 m/sec; support class 1.

Fig. 11 Rätan tailrace tunnel, section 2075.

Tunnel Unit 10s; diabase and fine-grained aplitic granite; heavily crushed and frequently altered to an earth-like material; slightly water-bearing; seismic velocity 4650 m/sec; seismic velocity ratio 0.73; support class 3 (Robot shotcrete before mucking).

TABLE 3 Seismic Velocity Data and Support Requirements for Rock Conditions along Tunnel Seismic Profiles

Rock Type	Rock Conditions	Seismic Velocity m/sec	SVR[1)] Tunnel	SVR Surface	Support Class
(1a) Coarse-grained Granite v_{max} = 5500 m/sec	Shear zone with altered seam, water-bearing, Unit 102	4500	0.82	---	2
	Blocky rock, some altered seams, Unit 10j	4500	0.82	---	2
	Shear zone with some open, water-bearing joints, Unit 10j	4500	0.82	---	2
(1b) Coarse-grained Granite v_{max} = 6000 m/sec	Blocky, loose rock around a one-meter-wide diabase vein, water-bearing, Unit 8	(4000/5200)[2)]	---	0.77	2
	Intersecting diabase and quartz veins in jointed rock, Unit 9	(3800/5200)	---	0.73	2
	Open, water-bearing seam in massive rock, Profile 303	4500	0.75	---	1
(1c) Coarse-grained Granite v_{max} = 6400 m/sec	Two 50-cm-wide vertical clay seams in blocky ground, Unit 5a	5900	0.92	---	2
	One-meter-wide, flat-lying sheared, altered seam in large to small blocky rock, partly altered; some open, water-bearing fissures, some seams gravel-filled due to "Rapakivi" alteration, Unit 5b	5000	0.78	0.56-0.74	3
	Two-meter-wide altered diabase vein in a shear zone, prominent structure is horizontal, Unit 5g	5600	0.87	---	2
	Shear zone with a one-meter-wide diabase vein and a talc seam, Unit 6	(4500/5600)	---	0.81	2

TABLE 3 - Continued 2

Rock Type	Rock Conditions	Seismic Velocity m/sec	SVR[1)] Tunnel	SVR Surface	Support Class
(2) Fine-grained Aplitic Granite v_{max}= 6400 m/sec	Shear zone, very heavily fractured rock in center of zone, Unit 2a	(3400/5000)	---	0.68	3
	An open, water-bearing seam (< one-meter-wide), Unit 10b	4000	0.63	---	2
	One to two-meter-wide altered seam, water-bearing, Unit 10d	4000	0.63	---	2
	Shear zone, see Fig. 9, Unit 10c	4000 (3500/5200)	0.63	0.67	3
	Shear zone, Unit 10k	4400 (3600/5200)	0.69	0.69	2
	Closely spaced vertical joints near amphibolite contact, Unit 10o	4000	0.63	---	2
	Closely spaced vertical joints, some water-bearing, Unit 10o	4000	0.63	---	2
	Shear zone, sugar-cube rock structure, some clay-filled joints, Unit 10r	4400	0.69	---	3
	Thrust zone, see Fig. 11, Unit 10s	4200-4650 (3800/5200)	0.66-0.73	0.73	3
	Thrust zone, intersecting diabase veins and seams of altered granite, Unit 10s	3700 (3700/4900)	0.58	0.75	3
	Blocky rock, see Fig. 10, Unit 10p	5500	0.86	---	2
	Closely spaced vertical joints, tight, Unit 11	6400 (3800/5200)	1	0.73	1
(3) Amphibolite v_{max}= 6100 m/sec	Very blocky rock, Unit 10g	5800	0.95	---	1
	Very blocky and loose rock around a talc-serpentine seam, Unit 10h	4000	0.66	---	2

TABLE 3 - Continued 3

Rock Type	Rock Conditions	Seismic Velocity m/sec	SVR[1] Tunnel	SVR Surface	Support Class
(4) Diabase v_{max} = 6300 m/sec	One-meter-wide fractured and altered vein; surrounding rock blocky and loose, Unit 3	(3400/5500)	---	0.65	2
	Large blocky rock, Unit 10e	5800	0.92	---	1
	Shear zone, Unit 10e	5000 (2900/5400)	0.79	0.54	2
	Large blocky rock near shear zone, Unit 10e	5400	0.86	---	2
	Shear zone, Unit 10f	4000	0.63	0.77	3
	Very blocky and loose rock around a chlorite-talc seam, Unit 10n	4300 (3800/5000)	0.68	0.76	2
	Shear zone, very blocky, Unit 10g	5200	0.83	---	2

1) SVR = Seismic Velocity Ratio

2) Seismic velocities in parentheses are surface values used to compute surface seismic velocity ratios. The values given are (velocity of weak zone/velocity of surrounding rock)

2.34 Seismic Velocity-Tunnel Support Relationship

It will be noted in Table 3 that the range of tunnel seismic velocity ratio values for all of the cases is from 0.58 to 1.0. The distribution of cases according to support class and tunnel seismic velocity ratio is shown in Fig. 12. A crude, but conservative, support classification based on the tunnel seismic velocity ratios from Table 3 and Fig. 12 is proposed as follows:

Seismic Velocity Ratio	Support Class
>0.90	1
0.8 - 0.9	2
<0.80	3

The limits of this classification should not be considered sharp, as the requirements for support are not well defined, and there exists considerable overlapping at the suggested boundary values of the seismic velocity ratios.

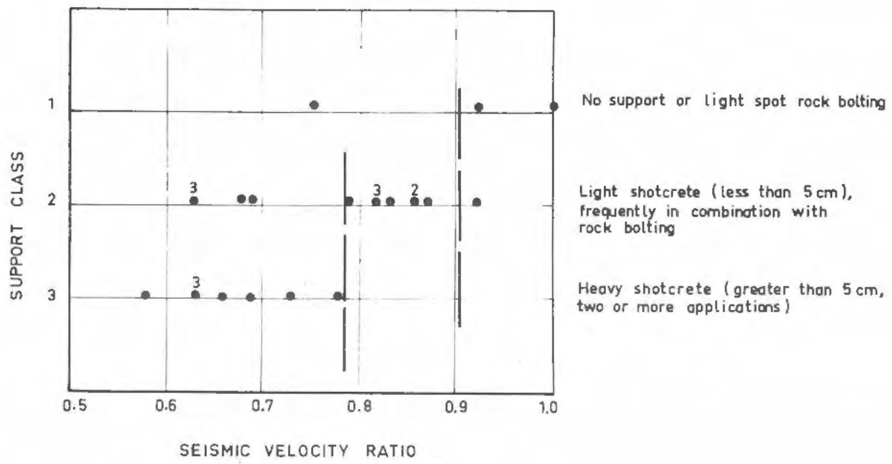

Fig. 12 Distribution of Rätan cases according to support class and seismic velocity ratio (tunnel seismic data).

For a more general classification, it is seen that the tunnel seismic velocity ratio value of 0.90 is a limiting value for the cases where rock support was used.

It was originally thought that a more refined classification for support measures could be established on the basis of the velocity ratio or some similar figure, but such refinements cannot be made. The following items explain why further refinements are not possible:

1. There is no purely objective way of determining the support requirements for rock tunnels. In Sweden, most rock conditions, even in large tunnels, can be supported with shotcrete or combinations of shotcrete and rock bolts. However, the selection of shotcrete thickness, rock bolt dimensions and spacings, and even the decision to shotcrete or not shotcrete, are made purely on the basis of judgment and experience. Hence, any classification of necessary support measures is inherently subjective, and refinements in different degrees of support are not possible.

2. A number of cases in which support is necessary have no characteristic velocity ratios. In particular, many of the large blocky structures, such as those found in diabases and amphibolites, are sufficiently intact that their seismic velocity ratios are high (> 0.90), yet their structural coherence sufficiently low that some form of support is necessary, usually rock bolting. In such cases, the support required to maintain stability is dependent on a number of factors other than rock quality, including size of tunnel, orientation of tunnel with respect to joint orientation, and joint surface friction. It is quite unlikely that any measureable quantity or parameter could be related to the support requirements in such rock.

The relationship between the support class and the seismic velocity ratios determined from surface refraction velocities is shown in Fig. 13 for the data available from the Rätan project. As will be noted in Table 3, some of the cases are instances where seismic velocities are known from both tunnel measurements and surface measurements, whereas others are instances where only surface data are available. From Fig. 13, it is apparent that it is not possible to make a distinction among the different support classes on the basis of ground surface refraction data. However, it is plainly evident that the ground surface seismic velocity ratio of 0.80 is a limiting value for the indication of weak bedrock zones that require some form of support during tunnel driving.

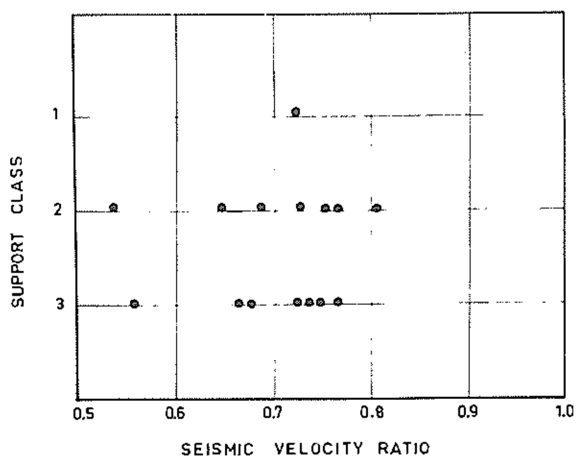

Fig. 13 Distribution of Rätan cases according to support class and seismic velocity ratio (ground surface seismic data).

From the tunnel seismic velocity ratio data it is seen that a seismic velocity ratio of 0.80 is an upper boundary for rock that requires heavy support (multiple shotcrete applications). The range of seismic velocity ratio of from 0.80 to 0.90 is thus a gray area in which correlation with support requirements depends on the manner in which the seismic velocities are obtained. When surface refraction data are being examined, it is suggested that areas with a velocity ratio in the 0.80 - 0.90 range be considered possible weak zones, whereas the data in Fig. 13 indicate that any areas with a velocity ratio of less than 0.80 be considered probable weak zones. The possible-probable designation does not have any connection with the degree of quality of the bedrock, but is rather intended only to quantitatively indicate the degree of probability of a weak zone of some sort existing along that particular section of a seismic line. This designation can be particularly useful if two or more parallel seismic lines are available and it is desired to map in weak zones in the bedrock. In the many cases where there may be a question as to the continuity or direction of a weak zone, the indication of probable weak zones and possible weak zones may be helpful in improving an uncertain or guessed interpretation. Furthermore, by using two indications for weak zones, one for probable and one for possible, the correct impression is conveyed that the degree of certainty associated with the interpretation of the velocity data is not 100 per cent. If only one indication for weak zones is used, the false impression may be given that there are definitely no weak zones in the bedrock other than those indicated.

2.35 Application of Results to Surface Velocity Data

In order to apply the velocity ratio concept to the location of weak zones from surface seismic velocities, it would seem necessary to know what type of rock lies under every meter of profile. Such information is seldom available and even in the most elaborate drilling investigations can only be guessed. In order to apply the information provided by Figs. 12 and 13, the Writer proposes the following guide to locate weak zones from ground surface seismic velocity profiles:

1. It can be said almost without exception that on the surface of the hard crystalline bedrocks of Sweden seismic velocities of 4000 m/sec or less are indicative of weak zones in the

bedrock. Stretches on a profile with such a velocity thus can be considered to be weak zones without further question, provided the surrounding higher velocities are greater than 4900 m/sec. In such cases, and in the cases to be subsequently discussed, the assumption is made that the low velocity zone is the same rock type as the immediately surrounding higher velocity rock. It will be shown below that this assumption, although not always correct, gives a realistic way of treating surface refraction velocities.

In the most common cases in Sweden a typical high quality bedrock has a velocity of at least 5000 m/sec. The 4000 m/sec zones thus correspond to a velocity ratio of 0.80, a definite weak zone that requires support according to the analysis of the tunnel measurements in Table 3.

2. Other weak zones can be located by comparing the magnitudes of all low velocity stretches on a profile with the average velocity of the surrounding higher velocity rock. In the simplest case, and a rather common one also, a low velocity zone is located in a relatively uniform velocity field in which most of the higher velocity values are of about the same magnitude. In such a case it is reasonable to apply the seismic velocity ratio concept to determine probable and possible weak zones.
Two examples from the seismic measurements over the tunnel are shown in Fig. 14. In (A) the average high velocity is taken as 5100 m/sec. The velocity limit for probable weak zones is taken as 80 per cent of 5100 m/sec, or 4100 m/sec. In (B) the corresponding values are 5800 m/sec and 4650 m/sec. In both of these examples the weak zones are shown as blackened bands along the profiles. From these two examples it can be noticed that one certain velocity is not always considered to be indicative of a certain quality of rock. In (A) the zone of 4500 m/sec is calculated as a possible weak zone, as it is 89 per cent of the average high velocity of the surrounding rock. In (B),

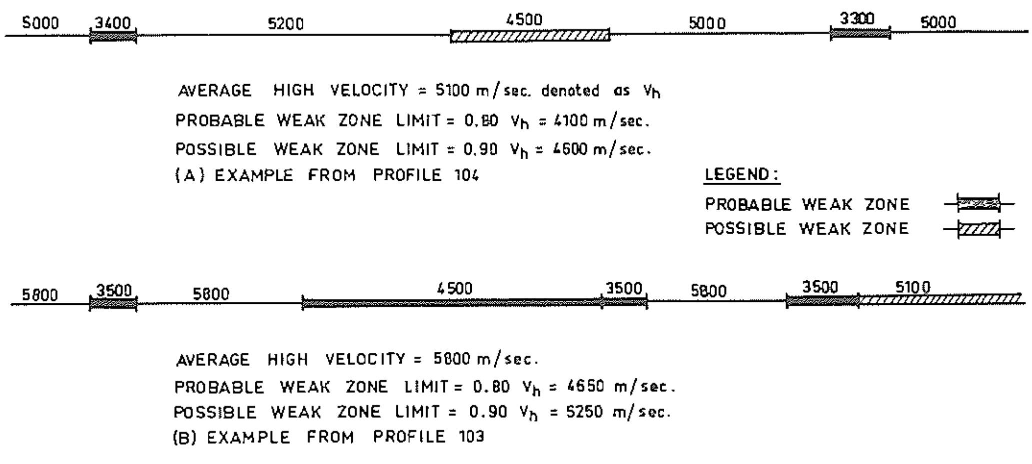

Fig. 14 Examples for interpretation of weak bedrock zones from surface seismic velocities.

however, the higher value of the average high velocity in the surrounding rock (5800 m/sec) yielded a higher limit for weak zones (4650 m/sec) and the long 4500 m/sec stretch falls into the probable weak zone category. In both of the above examples the interpretation for weak zones agrees well with the observations of rock conditions from the tunnel.

As pointed out earlier, the assumption has been made in the application of the tunnel data to the surface data that all of the weak zones in the bedrock border on, or are in contact with, a higher quality or higher velocity rock of the same type. Such an assumption is necessary if two adjacent velocity zones are to be compared. There are two reasons why the above assumption gives a valid indication of weak zones in the unweathered Swedish bedrock:

a) There are only a few instances in which a zone of one particular rock types is of low quality over its entire length. It is more common that a geologic unit, such as a large diabase intrusion, has one or more weak zones whose quality and velocity are below that of the average of the unit. This is true for all of the rock in the Rätan tunnel. In such cases the assumption is correct and the proposed procedure is consistent with the results of Table 3.

b) If an entire geologic unit is of a low quality, then its velocity is generally so low that it falls into the weak zone class for even the lowest velocity group, i. e. for the 5000 m/sec class rock types whose weak zone limit is about 4000 m/sec.

Thus in either of the above two cases it is likely that a comparison of adjacent velocity zones on the surface gives a realistic indication of weak zones. An example will be cited in order to clarify the above discussion. Units (e) and (f) in Zone 10 in the tunnel are all the same diabase unit. As can be seen from Profile 302 (Fig. 6) in the tunnel, the range of velocities in this section is from 4000 m/sec in the worst section (shear zone at (f)) to 6300 m/sec in the best sections (large blocky). The weak zone limit velocity is 80 per cent of the highest velocity, or 5000 m/sec. The corresponding low velocity zones are indicated along the tunnel in plan and profile. On the surface profiles above the tunnel there is no information which indicates the type of rock along this stretch. However, the low velocity zones have values equal to or less than 4000 m/sec and thus would be considered weak zones according to the first criterion. Even if the neighboring 5000 m/sec rock should be a different type of rock than the rock in the weak zones, the 4000 m/sec stretch would still be indicated as a weak zone, as its velocity value is less than 80 per cent of the average high velocity of 5200 m/sec in the adjacent stretches. The same case exists along most of the entire tunnel. For those cases in which the average high velocity values are above 5000 m/sec, such as above Zone 5, the velocity ratio concept has been used to indicate weak zones. The data used to compute the velocity ratio are shown in the figures. The velocity values for the high velocity rock have been taken as an average of the high velocities in the particular region under question.

2.36 Comparison of Velocity Data with Support Quantities

A summary of the support quantities and all velocity data is given in Table 4. The tunnel units listed in column one correspond to the designation on the drawings and the geologic descriptions given in the Appendix. The length of shotcreted tunnel section is given in the second column. These figures correspond only to the support work carried out during tunnel driving and do not include the extra shotcreting that was done after completion of the tunnel. The writer considers the former figures to be somewhat closer to the actual support that was necessary. Even these figures, however, include a good deal of shotcreting whose main purpose seems to have been a psychological protection for the miners.

The totals for Table 4 show that 1250 meters of tunnel have been shotcreted in full or part area. The total length of probable low velocity zones on the bedrock surface, according to the previously described method of interpreting velocity data, is 519 meters, or 42 per cent of the total shotcreted length of 1250 meters. Part of the difference in these two figures may be attributed to the existence of horizontally-oriented weak zones, such as the overthrust in Unit 12, that have no outcrop in the area immediately above the tunnel and hence have no corresponding low velocity zones from the measurements in the area.

A comparison of support quantities and velocity data for only those sections of tunnel in which seismic measurements were made is given in Table 5. The length of low velocity zones on the surface above Profiles 301-303 varies from 40 to 104 per cent of the total

TABLE 4 Support Quantities in Rätan Tunnel

Tunnel Unit	Length of Shotcreted Unit m	Most Probable Low Velocity Zone			
		Surface		Tunnel	
		Velocity m/sec	Length m	Velocity m/sec	Length m
1a	30	none[1]		-	-
1b	7	none[1]		-	-
1c	18	none[1]		-	-
2	60	3300-4000	20	-	-
3	5	3400	20	-	-
4	140	?		-	-
5a-5g	160	3200-4600	166	5000	20
5h	0	4500?	15	-	-
6	25	4500	10	-	-
7	1	none		-	-
8	5	4000	8	-	-
9	14	3800	20	-	-
10a	0	-	-	-	-
10b	4	none		4000	5
10c	5	3500	12	4000	5
10d	11	none		4000	5
10e-10f	110	2900 2900	11 6	5000 4000	15 5
10g-10h	85	4000 3500?	66 4	4400 4000	13 5
10i	0	none		none	
10j	42	3500	11	4500	30
10k	10	3600	8	4400 4700	10 20
10l	4	none		4500	10
10m-10n	62	3500	15	4300	13
10o	72	2800 3200	8 7	4000	10

(continued)

1) These weak zones are probably the low velocity zones on Profiles 1 and 6, 100 meters to the south of the tunnel.

Table 4 (continued)

Tunnel Unit	Length of Shotcreted Unit m	Most Probable Low Velocity Zone			
		Surface		Tunnel	
		Velocity m/sec	Length m	Velocity m/sec	Length m
10q-10s	145	3700	45	4400	20
		3800	29	4650	35
		3700	15	4200	15
		3400	9	4800	10
				4000	4
				4700	11
				3700	10
				4000	10
				5000	15
10t	50	3500	6	-	-
		3800	8		
11	0	4000	11	-	-
12	180	none		-	-
13	5	none		-	-
Total	1250		519		

$$\frac{\text{Total length of low velocity zone on surface profiles}}{\text{Total length of shotcrete support in tunnel}} = \frac{519}{1250} = 42\ \%$$

length of shotcreted tunnel for the corresponding weak zones in the tunnel. The average for all of these zones is 55 per cent. The same average figure for the tunnel measurements (compared to the total shotcrete length over the measured profiles) is 44 per cent.

The predominantly high velocities on tunnel Profile 301 in Unit 5 can be attributed to the predominant horizontal orientation of the major weaknesses in this rock. Horizontal seismic pulses can travel practically unhindered through such horizontally stratified or fractured rock, in spite of the discontinuous nature of the rock in the vertical direction. Pronounced vertical discontinuities, such as at (b) in Unit 5, do, however, disrupt the seismic wave path and result in low velocities. Unit 5 dips rather steeply, as seen in cross-section in Figs. 4 and 5. The extensive low velocity zones at the bedrock surface above Unit 5 are probably due to an unusually high degree of weathering and loosening.

5 Comparison of Support Quantities and Seismic Velocities for Rätan Tunnel

Number	Length of Shotcreted Tunnel, m (1)	Length of Low Velocity Zone, m Tunnel (2)	Surface (3)	$\frac{(2)}{(1)}$	$\frac{(3)}{(1)}$
it 5	160	20	166	0.12	1.04
it 10	550	276	222	0.50	0.40
	0	13[x]	10	-	-
	710	309	388	0.44	0.55

low velocity zone is the result of one open, water-bearing fissure in
sive granite. The feature has not had any effect on the stability of the tunnel.

Although the above statistics point out that there is no unique relationship between total length of low velocity rock and shotcrete requirements, the seismic velocities do have one positive property: every low velocity zone on the surface profiles above the tunnel, with one exception, has a corresponding weak zone in the tunnel that required support of some type. The ratio of the length of low velocity zones on the surface profiles to the length of the corresponding supported weak zones in the tunnel is between 42 per cent (for the entire tunnel length) and 55 per cent (for only the sections of tunnel along Profiles 301-303). The fact that these ratios are not 1:1 or 100 per cent can be explained by the following:

(a) Some of the rock that requires support in the tunnel is relatively high quality rock that has a high seismic velocity ratio. The need for support in such rock may arise out of such factors as adverse joint orientation and intersection, neither of which usually has a large affect on seismic velocity.

(b) The support requirements in a tunnel cannot be evaluated in any objective way. The personal judgement used in the application of shotcrete leaves room for wide margin in the determination of the required extent of shotcrete. In the Writer's experience the most common tendency in this exercise of judgement is towards an excessive use of shotcrete, particularly when a private shotcrete contractor is engaged. Furthermore, a good deal of shotcrete is used solely for the protection of miners against small pieces of falling rock and is not necessary for the maintenance of stability.

2.4 Discussion of Supplementary Data

2.41 Bedrock Surface Profiles

Although the seismic velocities provide the most positive information about weak zones, the bedrock surface profiles can be helpful for indicating the approximate extent of weak zones. If the bedrock profile is plotted on an exaggerated horizontal-vertical scale, such as in Fig. 4, the bedrock topography is distorted to such a degree that all ravines and low spots on the bedrock surface are plainly visible. Although ravines on the bedrock surface cannot be associated with 100 per cent certainty to weak zones, their presence is a definite reason to suspect that a weak zone may exist. Low velocity zones often are coincident with ravines or bedrock surface depressions. In such cases the existence of a weak zone is almost certain, and further investigation with diamond drilling is justified if additional information about the nature and extent of the weak zone is desired.

2.42 Destressed Rock at Tunnel Periphery

The tunnel refraction measurements were carried out with a normal geophone spacing of 5 meters. Several lines were shot with spacings of 2.5 meters and 1 meter. As reported by Craelius (Craelius Terratest AB, 1967), the thickness of the detectable low velocity zone for all of the measurements is only 1-1.4 meters. In consideration of the 0.5-1.0-meter thick layer of crushed rock and tunnel muck that lies on the tunnel floor, it must be concluded that the thickness of any destressed or loosened zone along the bottom of the tunnel walls is less than one meter. There are two possible interpretations for this results:

1. The thickness of one meter given by the fraction measurements is the true thickness of the destressed zone at this point. Such a case would be somewhat consistent with the results reported by Rahm (Rahm, 1965) for an inspection tunnel under Trängslet dam and for a tailrace tunnel at Västra Frölunda, Gothenburg. Measurements with a seismosond and with ultrasonic equipment in these tunnels showed a 0.25-0.75-meter-thick zone of low velocity rock at the periphery of the tunnels. Although these latter measurements were made with an entirely different technique and in tunnels with an area of less than one-tenth of the Rätan tunnel, they do indicate that no exceptionally deep stress relieving has taken place at the tunnel periphery. The fact that the thickness of the low velocity layers is of the same order of magnitude in these tunnels of different areas may indicate that the destressing is due primarily to blasting and is not so dependent on tunnel size as one might expect. Similar results have been obtained from seismic refraction measurements in granite tunnels in the United States. The absence of thicker destressed zones in these tunnels has been partly attributed to the effectiveness of rock bolts in preventing loosening of rock at the tunnel periphery. In Rätan the measurements were made in the floor of the tunnel where there was no rock support. The fact that a relatively thin shotcrete layer (10 cm) is usually sufficient to maintain roof stability in Swedish tunnels, even in extremely low quality rocks, would further lead one to believe that the layer of destressed rock at the tunnel periphery is relatively thin.

2. It is quite possible that the measured low velocity layer is only the outermost layer of a thicker zone whose degree of destressing is too small to influence its seismic velocity. In such a case the destressed zone itself may act as a refracting medium, as its velocity would be the same as that

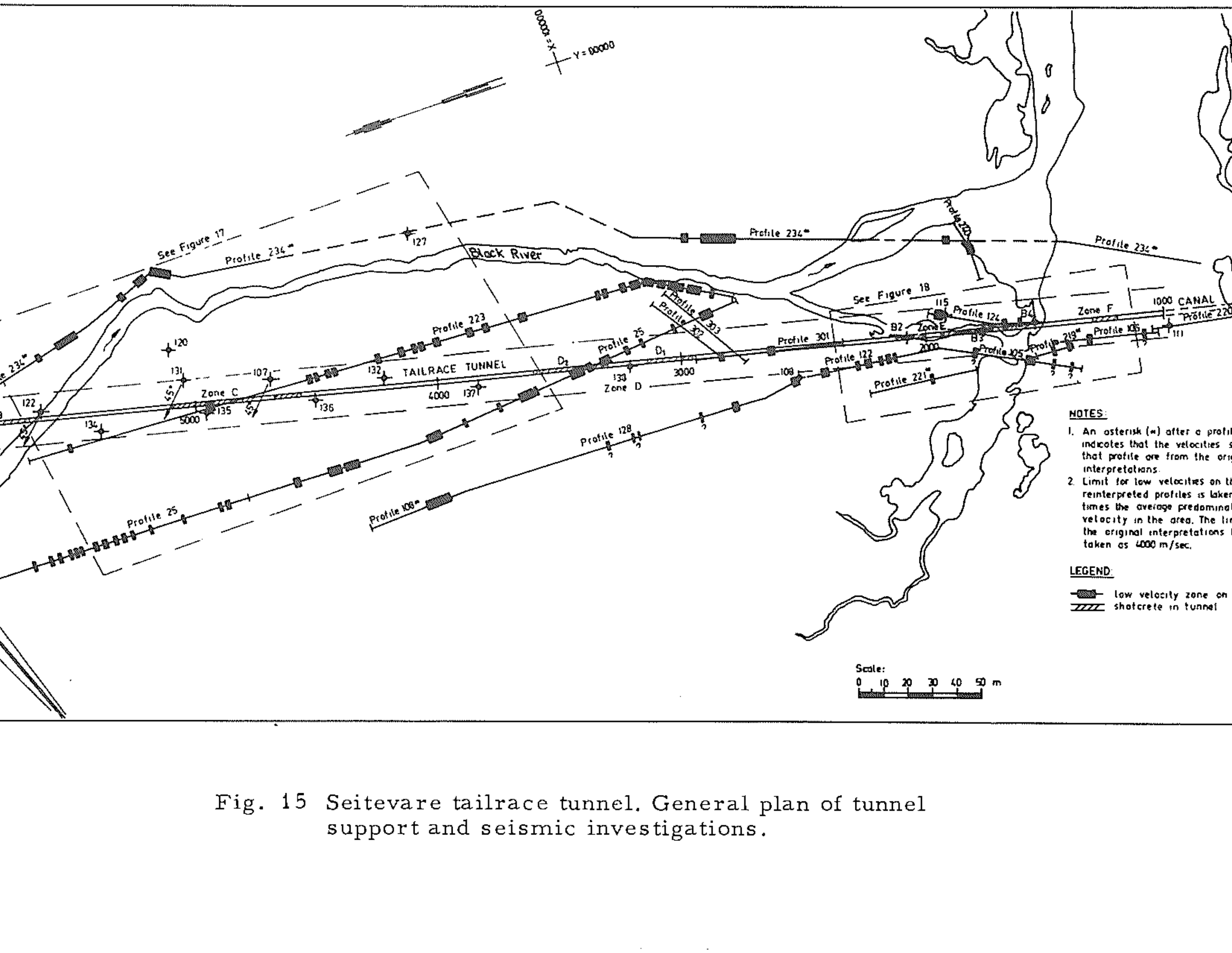

Fig. 15 Seitevare tailrace tunnel. General plan of tunnel support and seismic investigations.

of undisturbed rock. The influence of such a tight destressed zone on the stability of a tunnel is not known, but it would not seem likely that such a mildly relieved zone would itself require any reinforcement.

The above discussion is intended only to show two possible interpretations of the seismic data. A more detailed measurement program would be necessary before any concrete conclusions could be drawn concerning the extent of the destressed zone for large tunnels. Measurements should be aimed at determining the stress relief profile around the entire periphery of a tunnel section.

3. APPLICATION OF RÄTAN RESULTS

3.1 Seitevare

A general plan of the Seitevare tailrace tunnel is shown in Fig. 15. Extensive surface refraction investigations were carried out at the site by ABEM[1] in 1958. All of the refraction profiles in the vicinity of the tunnel have been interpreted in the same manner as those at Rätan, and hence the criteria derived from the latter can be applied to the Seitevare investigations. A few of the original seismic profiles that were not interpreted in as great a detail as the more recent profiles shot along the final tunnel line are marked with an asterisk in the figures.

The bedrock at Seitevare consists primarily of precambrian granite and lesser amounts of leptite, a metamorphosed volcanic sediment. The velocity profiles are relatively simple with respect to variations due to different rock types. Because the dominant high seismic velocities over most of the area are about 5000 m/sec, the weak zone limit is 4000 m/sec. Unless otherwise noted in the figures, all weak zones (indicated by black bands on the profiles) have velocities equal to or less than 4000 m/sec.

Detailed views of the seismic profiles and the individual zones of poor rock are given in Figs. 16-19. Those portions of the tunnel that have been shotcreted are cross-hatched. A question mark (?) at such a zone means that no weakness zones are visible in the tunnel walls.

1) Aktiebolaget Elektrisk Malmletning.

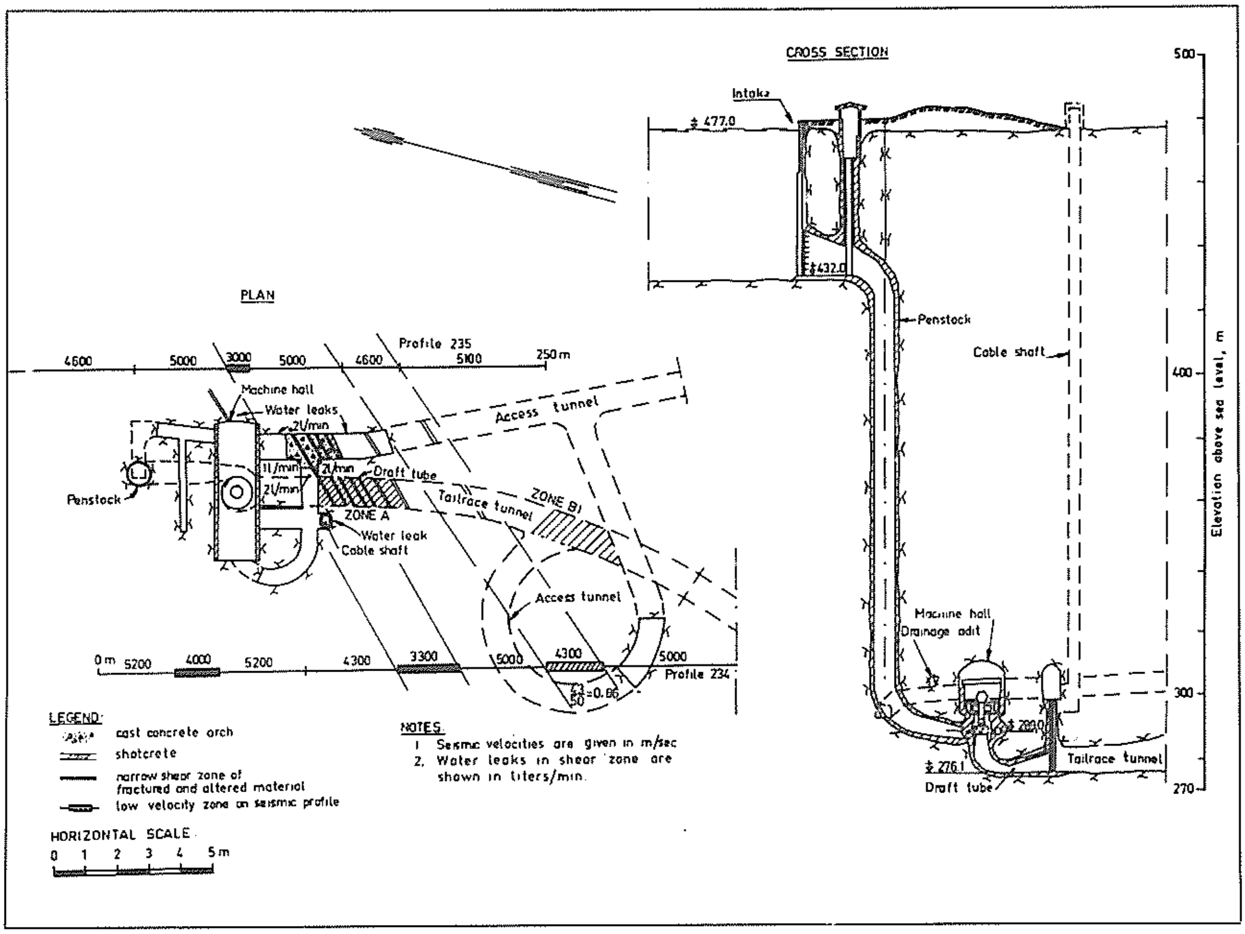

Fig. 16 Seitevare machine hall area.

Short descriptions of the individual weak zones that have been observed in the tunnel walls are given below. The figures for comparison of support quantities and seismic data are given in Table 6.

ZONE A - A series of eight narrow (20-50 cm) vertical shear zones of fractured and altered granite. These features are intersected in the access tunnel at the entrance to the machine hall, where they have been treated with a 13-meter-long cast concrete arch, and again in the draft tube, where they have been supported with shotcrete and shotcrete arches. The shear zones coincide well with two low velocity zones directly above the area (see Fig. 16).

ZONE B - No weak zones are visible in the long stretch near the river. At B1, shotcrete reinforcement has been applied over a local inclusion of leptite. The projection of this zone to the surface is not certain. The zone may correspond to one of the two low velocity zones on profile 234 that lie to both sides of Zone A.

Comparison of Support Quantities and Seismic Velocities for Seitevare Tunnel

unnel	(1) Length of shotcrete deemed necessary for tunnel stability m	(2) Length of shot-crete actually used m	(3) Corresponding low velocity zone on bedrock surf Velocity m/sec	Length m
	25	25	3300	20
	20	20	4000 ?	20
aining)	-	290	-	-
	100	460	3700	18
	200	500	4000	210
	80	380	2500-4000	53
	20	100	4300	22
	445	1775		343

$$\frac{\text{TOTAL (3)}}{\text{TOTAL (2)}} = \frac{343}{1775} = 19\%$$

$$\frac{\text{TOTAL (3)}}{\text{TOTAL (1)}} = \frac{343}{445} = 77\ \%$$

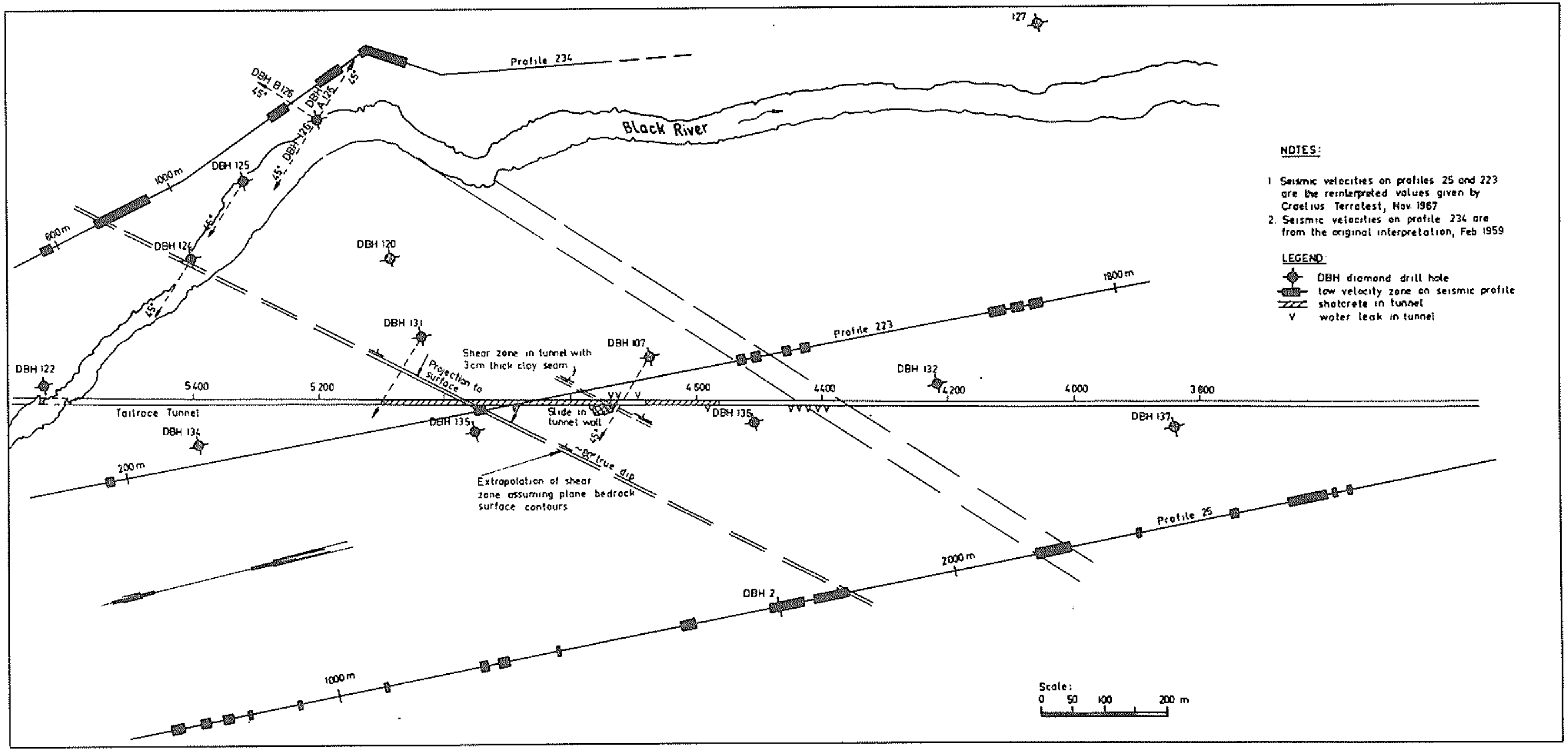

Fig. 17 Seitevare tailrace tunnel, zone C.

ZONE C - A leptite inclusion with 8 to 10 isolated clay seams, clay-filled joints, and mineralized slickensides that have led to minor falls and overbreak. One three-cm-thick clay seam that occurs in a shear zone has caused a small slide in the tunnel wall (see Fig. 17). No extensive weak zones occur in Zone C, and most of the shotcrete has apparantly been used as a protective measure against smaller pieces of falling rock. The only low velocity zone on the surface directly above the tunnel is the short 3700 m/sec zone that corresponds to the clay seam that caused the wall slide. A group of several short low velocity zones just to the south of Zone C correspond to four or five open, water-bearing fissures in the tunnel at that location, marked with V in Fig. 17.

ZONE D - Two stretches of soft granite with widely spaced, steeply dipping, slickensided, clay-coated joints. Rock bolting and shotcrete have been necessary to avoid larger roof falls and wall slides. The rock in the upstream section, marked D2 in Fig. 15, is much softer and looser than that in D1, and has required more support. No pronounced weakness zones are visible in the walls of the D1 section. As seen in Fig. 15, the D2 section corresponds to the wide band of low velocity rock on the bedrock surface that stretches across the area in a southeast direction. No low velocity zones on the bedrock surface correspond to the D1 section.

ZONE E - A 15-meter-wide overthrust zone with heavily jointed and partially altered granite (see Fig. 18). Clay seams and chemical alteration products are very frequent in this section. Shotcrete has been used for support. As seen from Figs. 18 and 19, the overthrust corresponds to low velocity zones on Profiles 122 and 126. The two downstream sections of Zone E, indicated with question marks on the drawings, have no visible weak features in the walls of the tunnel.

ZONE F - A leptite inclusion with some clay-filled joints. The zone possibly corresponds to the low velocity zone on Profile 106. Support in the roof is primarily for safety purposes.

From Table 6 it is seen that the total length of the velocity zones that correspond to the major weak zones in the tunnel is about 77 per cent of the length of shotcrete support that is considered necessary for tunnel stabilization. If all the shotcrete is considered, the same ratio is about 19 per cent. These figures have been tabulated with a knowledge of the nature and extent of the weak zones in the tunnel and the corresponding low velocity zones on the bedrock surface. This information is not available at the preliminary investigation stage. The seismic velocity data can, however, be used to make an estimate

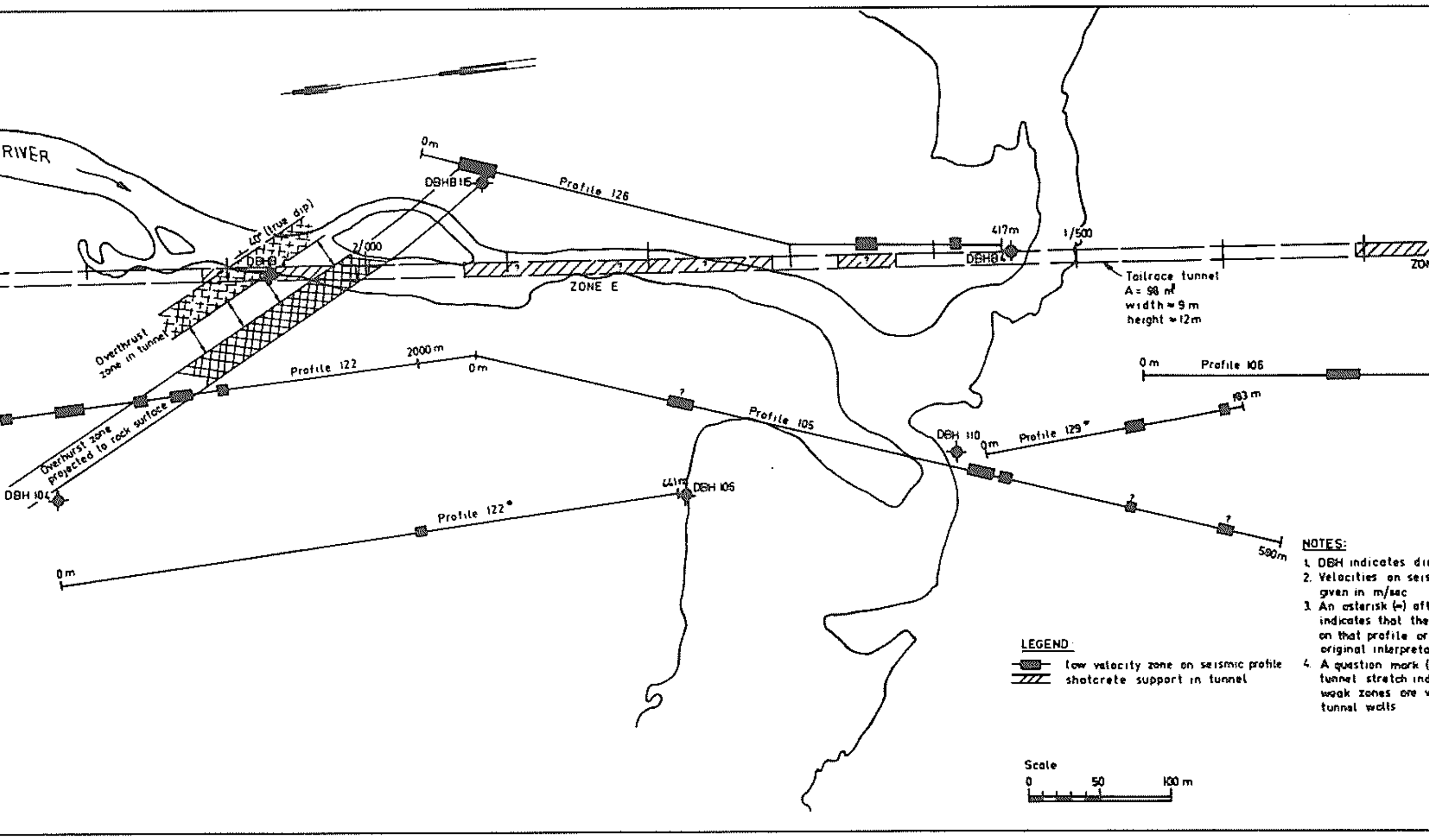

Fig. 18 Seitevare tailrace tunnel, zones E and F.

of the minimum support requirements. At the preliminary investigation stage one would naturally have to consider all low velocity zones as potential weak zones. Because of the sudden changes in rock conditions that often occur over short distances, it is suggested that only those low velocity zones within 100 meters distance of the tunnel be considered. In Fig. 15 these limits are shown with two dashed lines that run parallel to the tunnel. The total length of low velocity zones within this area is about 575 meters.

This includes <u>all</u> low velocity zones within the 200-meter-wide band. Some of the low velocity zones in this area are probably only surficial weak zones that do not extend below the upper five meters of the bedrock surface. Others may be zones that are continuous to the depth of the tunnel, but are so tight that they do not require any support in a tunnel. Furthermore, some of the low velocity zones with the band are probably the same weak zone that occurs in parallel profiles and hence should not be counted twice. This case may be obvious if a weak zone cuts across the tunnel at right angles, and should be taken into consideration when weak zones are being estimated. The

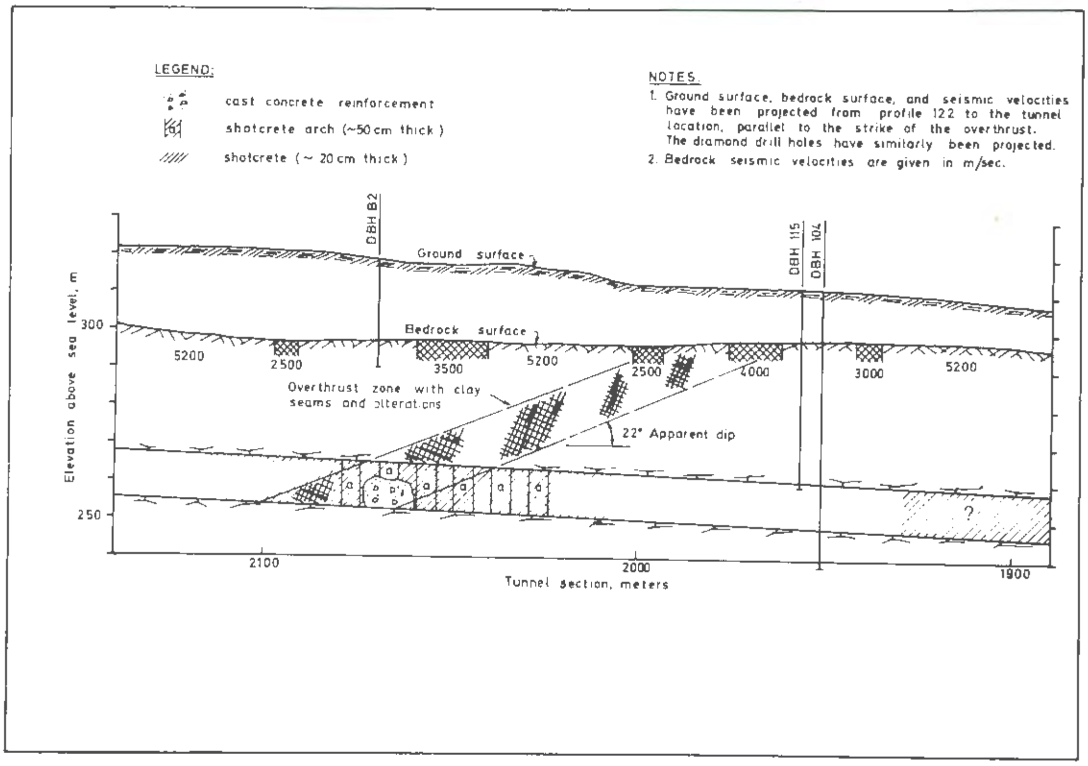

Fig. 19 Seitevare tailrace tunnel, zone E.

first two uncertainties concerning the continuation of a surfical weak zone to the depth of the tunnel can be investigated with a slight extension of the normal refraction technique. According to Craelius Terratest, the existence of weak zones at greater depths (up to 50 meters) can be checked by shooting extra shot points at long distances from the end points of the geophone spread. With this additional information about velocities at the tunnel level, one would be in a much better position to estimate the magnitude of potentially poor rock.

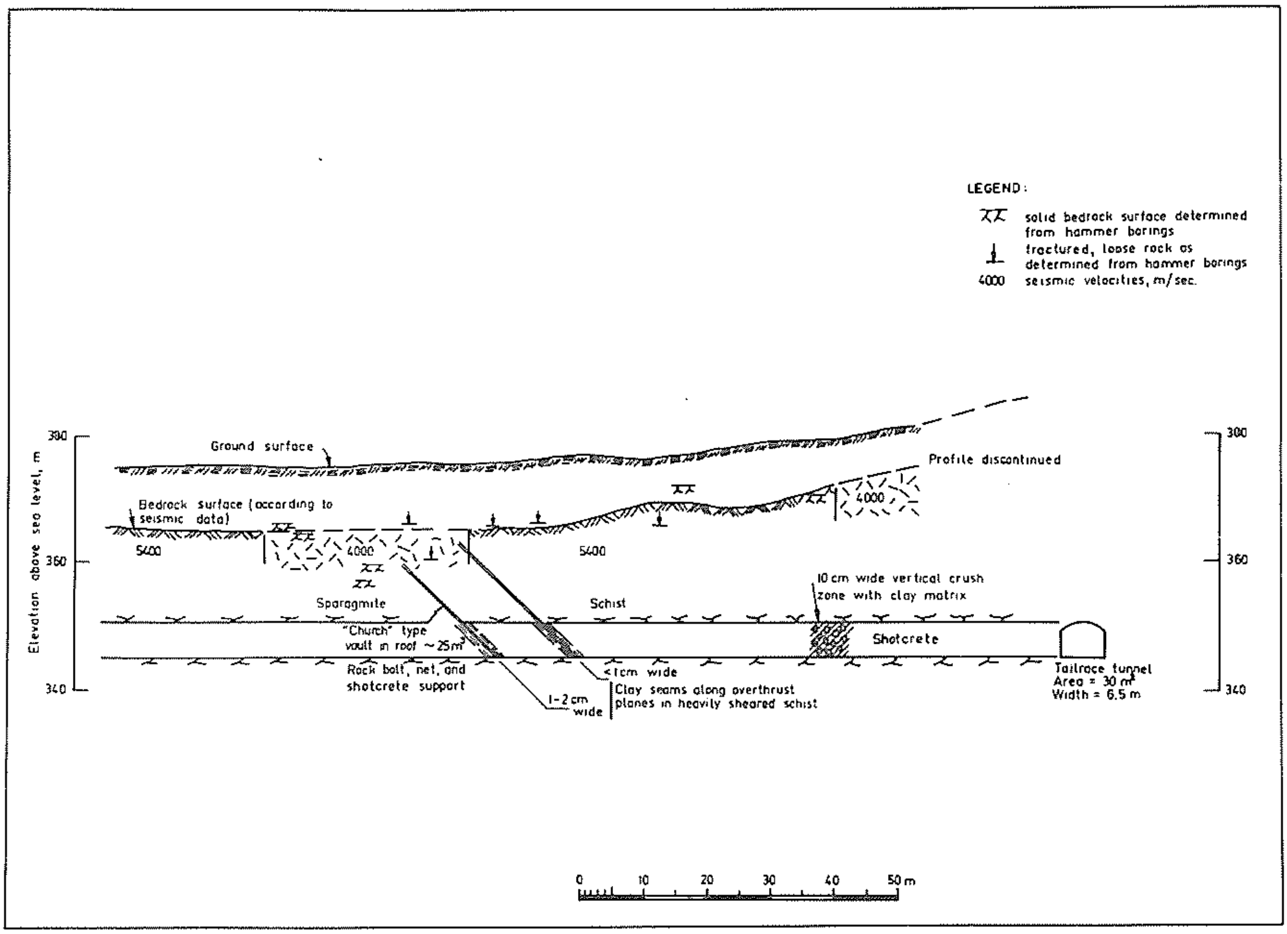

Fig. 20 Bergvattnet tailrace tunnel. Bedrock weak zones in downstream section.

If the total length of low velocity rock within the 200-meter-wide band is considered a minimum for the length of shotcrete support necessary, one might have estimated that about 600 meters of the Seitevare tunnel would require support. Although admittedly a crude approximation, this figure is certainly more realistic than the indications of poor rock in the 14 core borings that were made along the tunnel line.

3.2 Bergvattnet

A cross-section of a short stretch of the tailrace tunnel at Bergvattnet power station is shown in Fig. 20. The bedrock at this particular section on the tunnel is schist and sparagmite. Weak zones of 4000 m/sec are shown on a seismic profile that was shot directly over the tunnel. One 32-meter-long 4000 m/sec zone corresponds to two over-thrust zones in a heavily sheared schist. Thin (less than 3 cm wide) clay seams in these zones have caused a roof fall (about 25 cubic meters) at one section and have threated to cause further trouble through a washingout of clay and other fine joint-filling material. At another location about 500 meters upstream, a 4000 m/sec zone at the end of the same profile corresponds to a 10-cm-wide vertical crush zone with a clay matrix.
The softening and washing out of the clay at this section that started under the construction period would probably lead to roof falls if the process is allowed to continue.

The above examples point out the particular strong significance that can be attatched to seismically determined weak zones when the rock cover is very thin.

4. CONCLUSIONS

It has been possible to draw the following conclusions from the seismic refraction studies in Rätan, Seitevare, and Bergvattnet:

1. Longitudinal seismic velocity is a good measure of rock quality in the hard, unweathered crystalline bedrocks of Sweden.

2. The longitudinal seismic velocity is dependent on the properties of the intact rock. This fact has been taken into account in the analysis of the tunnel data by normalizing velocity values according to rock type. The seismic velocity ratio is a normalized "velocity index" and has been defined as the ratio of the longitudinal or primary seismic velocity in a particular rock mass to that of the intact or massive rock of the same type. Both velocities are determined from in-situ refraction measurements.

3. A conservative classification for support requirements in medium to large tunnels (> 30 sq. meters in cross-sectional area) in the hard, crystalline bedrocks of Sweden has been derived on the basis of the tunnel seismic velocity ratio data. The proposed classification is as follows:

Seismic Velocity Ratio	Support Requirement
>0.90	none to spot rock bolting
0.80 - 0.90	light shotcrete (< 5 cm), frequently in combination with rock bolting
< 0.80	rock bolting, heavy shotcrete (> 5 cm, two or more applications)

This classification is to be taken only as a crude guideline. Exceptions to the system can be expected, particularly for large blocky ground that requires pattern rock bolting and for rock containing thin, widely-spaced clay seams.

4. A more general designation for rock quality and support requirements is to consider rock with a seismic velocity ratio of less than 0.8 as a weak rock that requires some form of support, usually at least 5 cm of shotcrete.

5. Not all of the rock that requires support in a tunnel has a characteristic low seismic velocity. In particular, adverse joint orientations and intersections that can cause roof falls and wall slides do not usually have any affect on the seismic velocity.

6. The seismic velocity ratio concept can be used to identify weak zones from surface refraction measurements by applying the weak rock criterion to the average high velocity values along surface profiles. Velocity ratios less than 0.8 can be considered characteristic of <u>probable</u> weak zones. Those in the 0.80-0.90 range can be considered characteristic of <u>possible</u> weak zones.

7. There is no unique relationship between the length of shotcrete support for a tunnel and the length of low velocity rock. A crude estimate for the minimum shotcrete quantities <u>for those weak zones indicated by the seismic measurements</u> can be taken as a length equal to the total length of low velocity zones on the surface profiles. Because of safety precautions and disputable support requirements, the real support quantities may be twice

those indicated by the seismic measurements. The support necessary for flat-lying weak zones cannot be estimated from seismic refraction data. Nor can any estimates for specific bolting requirements be made from seismic data.

8. For shallow tunnels (less than 100 meters of rock cover) in the hard, unweathered bedrocks of Sweden, surface seismic refraction measurements along a tunnel line give a more realistic indication of potentially poor rock conditions than do randomly located drill holes.

It has been demonstrated with the help of the Seitevare and Bergvattnet projects that the concepts developed at Rätan are generally applicable to the hard, unweathered crystalline bedrocks of Sweden. Applications in completely different geologic environments, such as sedimentary rocks or totally different igneous environments that are extremely weathered or altered, should not be made at this time. Furhter case studies will naturally give a better insight into the general applicability of the Rätan data, but for the present time general applications to all types of geologic environments are not possible. The results, however, should be applicable to similar geologic environments in other parts of the world, such as on the Canadian shield. It is also likely that criteria similar to those developed at Rätan could be developed for other geologic environments.

The results from the Rätan work give only an improved interpretation of the magnitudes of seismic velocities. The problem of projecting the location of weak zones from the bedrock surface down to the level of the tunnel still exists. Tectonic mapping of surface geologic features, such as was done at Rätan, combined with geologic mapping of weak zones in the tunnel, can be used to establish the orientation (strike and dip) of major weak zones at an early stage of the project. Provided pronounced structural trends exist in the bedrock, this information can be used to extrapolate seismically determined weak zones down to the tunnel level.

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APPENDIX

Geologic Descriptions of Tunnel Units, Rätan

Section m	Tunnel unit	Tunnel support classification x)	Geologic description (after M Bekkelund, Sydsvenska Kraft AB)
Downstream side			
0-120	1a	3, CA	Amphibolite altered to earth-like material.
	1b	3	Heavily sheared granite.
	1c	3, SA	Diabase-structure varies from thin slices to large blocks to heavily altered material.
157-219	2	2	Shear zone in fine-grained aplitic granite.
194-201	2a	3, SA	Particularly heavily fractured and loose.
450-452	3	2	Altered and sheared diabase vein, water-bearing.
550-650	4	3	Jointed and loose granite in the vicinity of two thick (5-30 cm wide) clay seams that run parallel to the roof.
812 and 826	5a	2	Two vertical clay seams with jointed and loose granite on both sides.
840	5b	3	Weathered seam with large and small blocky loose granite.
850-970	5c	2	Small blocky granite with crushed rock in open, water-bearing joints - predominant horizontal structure.
882	5d	2, SA	Vertical seam with crushed filling material.

x) Support Classification

1 None to spot rock bolting
2 Light shotcrete (<5 cm), frequently in combination with rock bolting.
3 Rock bolting, heavy shotcrete (>5 cm, two or more applications)
SA Shotcrete arch
CA Cast concrete arch

Section m	Tunnel unit	Tunnel support classification	Geologic description
890-960	5e	2	Altered granite with water-bearing seam
893 and 906	5f	2, SA	Two vertical, open, water-bearing seams.
975	5g	2	Weathered diabase vein, 1-2 meters wide, water-bearing.
1055	5h	1	Two narrow (50 cm) diabase veins.
1200-1240	6	2	Granite shear zone with a 1-meter wide diabase vein at 1200 and a talc-serpentine seam at 1216-1220.
1250-1260	7	2	Altered chlorite seam in roof with fractured granite in floor.
1310	8	2	A 1-meter wide diabase vein, water-bearing.
1388	9	2	Narrow diabase vein with jointed granite on both sides.
1457	10a	1	A 3-meter wide diabase vein, tight.
1462-1479	10b	1, 2	Three open vertical seams in fine-grained granite.
1500	10c	1, 2	Weathered, water-bearing seam in fine-grained granite.
1519-1529	10d	3	Heavily crushed fine-grained granite seam.
1529-1589	10e	1-2	Diabase-- large blocks and slabs.
1590-1615	10f	3	Diabase shear zone.
1638-1710	10g	1	Large blocky amphibolite.
1653-1679	10h	2	Sheared amphibolite with a talc-serpentine-chlorite seam.
1710-1745	10i	1	Transition from amphibolite to a weaker granite.
1745-1785	10j	2	Loose large blocky granite with a heavily altered seam at 1747-1751.
1790-1815	10k	2	Narrow, vertical slabs of fine-grained granite.

Section m	Tunnel unit	Tunnel support classification	Geologic description
1815-1850	101	2	Massive granite with a small shear zone at 1845.
1850-1878	10m	1	Large blocky diabase.
1878-1910	10n	2	Large blocky amphibolite with heavily fractured and loose rock around a chlorite-talc seam at 1870.
1910-1982	10o	1 locally 2	Fractured fine-grained granite with a shear zone at 1910-1930 and 1941-1970, particularly water-bearing joints at 1953-1960.
1982-2021	10p	1	Coarse-grained granite with water-bearing joints at 2003-2020.
2021-2046	10q	2	Diabase with several sheared areas and a few open water-bearing joints.
2046-2061	10r	3	Fine-grained "sugar-cube" granite with clay-filled joints.
2061-2176	10s	3	Alternating layers of weathered "sugar-cube" diabase and fine-grained granite. Water-bearing joints and some clay seams - the worst rock in the tunnel
2225-2255	10t	1 locally 2	Large blocky amphibolite, several open seams.
2370-2400 2470-2480	11	1	Two vertically-jointed aplitic granite zones, joint spacing 10-20 cm, tight rock.
2530-break-through 740-break-through upstream	12	2	Overthrust zone in granite horizontally fractured rock with open joints and frequent clay seams.
Upstream side			
680-720	13	1	Two diabase veins with a wide water-bearing vertical joint at 703.
580-640	14	1	Several relatively flat-lying overthrust joints which have caused minor overbreaking.

Section m	Tunnel unit	Tunnel support classification	Geologic description
280-350	15	1	Overthrust zone in granite with a vertical diabase vein at 287. Joint spacing 15 cm-1 m. Limited overbreak along a few clay-filled joints.

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PROBLEMS WITH SWELLING CLAYS IN NORWEGIAN UNDERGROUND CONSTRUCTIONS IN HARD-ROCKS

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PROBLEMS WITH SWELLING CLAYS IN NORWEGIAN UNDERGROUND CONSTRUCTIONS IN HARD-ROCKS

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Introduction

It is desirable that tunnels and other underground constructions in hard rock should be cheap, maintenance free, dry and stable without any need of support. Such ideal underground openings exist, but very seldom.

In Norway engineering-geologist is used to predict the extent of difficulties that may occur in the underground constructions, their type and dimensions and how to avoid or reduce them. His recommendations have, in general, been based on personal experience. But as time goes by, an increasing number of problems can be analyzed and measured. It appears that in the future it will be possible to give more objective and valid forecasts for many important problems.

In general we can differentiate between problems of blasting and advance and those relating purely to stability, although the stability problems can, of course, also be classified as problems of advance. The blasting problems in Scandinavia are often those of high resistance to blasting, poor drillability and high water leakage. Blasting and advance problems will not be discussed here.

The stability problems, in the form they are met with in underground constructions in hard-rock in Scandinavia, are more or less influenced by the local water conditions. They may be divided into the following five main groups:

1) Problems which are caused by too high and anisotropic rock stresses in relationship to the tensile strength of rock material. These cause rock burst phenomena or cracks and deformations which also may lead to overbreaks.

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1) Lecture to the Swedish Geotechnical Society, February 2, 1970.

2) Problems caused by chemical processes, frost action and perma-frost.

3) Problems caused by loosely jointed rock which is not sufficiently restrained by the adjacent rock masses by arching effect or by lateral pressure.

4) Problems which are caused by incorrect drilling or blasting.

5) Problems which are caused by clay zones in the rock. This problem will be dealt with below.

From experience, approximately 75 percent of the total cost of support in underground constructions in Norway is due to clay zones. Such zones are also the cause of most of the large slides and over-breaks which occur during the construction period and the subsequent use of the tunnels. This problem is encountered more or less every-where in Norway, but the extent, and thus the costs, of support are, of course, dependent on the aim of the construction.

In tunnels for hydroelectric powerplants where it is sufficient to prevent slides and where small amounts of subsequent overbreaks can be permitted, the costs of support of clay zones vary generally be-tween 8 and 15 percent of the pure excavating costs. This holds also for very large jobs when the area of the tunnel is small. In some cases the costs have been much less, but sometimes the costs of support with respect to clay zones have been up to 100 percent of the costs of excavation. The costs are frequently much larger for traffic tunnels because of the rigid requirements against overbreak. In such cases the expenses may be up to 400 percent of the costs of excavation. The costs of pavement, ventilation, light and archi-tectural requirements are not included.

The location and the orientation of the individual rooms of large con-centrated underground constructions can frequently be adjusted to fit the local geological conditions. The costs of support are therefore much lower than for large tunnels. The location of these latter is as a rule more or less fixed. If by oversight a clay zone or small clay zones with an unfavourable orientation are located within one of the rooms, the costs of supports have been the dominating cost. In Norway the cost of supporting clay zones to prevent slides is at the moment approximately 50 million Norwegian crowns a year in spite of the effort spent to reduce these difficulties. In addition to these come the considerable costs for delay caused to supporting works. (The sum is perhaps more an indication of the magnitude and the number of underground works in Norway than of character of the rock.)

The clay zones and their stability problems

A large clay-bearing fault zone is shown in figure 1 as it appears at the surface when the zone is oriented perpendicular to the direction of the movement of the ice sheet during the quaternary glaciation. In general it is possible to locate these zones on aerial photographs, and transfer them to topographical maps. The location of a fault zone and its intersection with a proposed tunnel is shown in figures 2 and 3. The location of the zones below the rock surface can be calculated by assuming that the fault zone is a plane. If the ground surface is flat, it is necessary to determine the dip of the fault zone by diamond drilling. If the surface of the rock is covered with soil, it is often advantageous to combine diamond drilling with seismic profiles in order to locate the clay zones and their orientation. Often it is necessary to make a geological map of the area in order, among other things, to study the jointing structure in detail, particularly in areas with thrust masses.

Figure 4 shows a clay zone in a tunnel. There is no leakage in the vicinity of the zone and only a small amount of overbreak.

In figure 5 is shown a tunnel through a clay zone where a little water leakage was noticed. The picture has been taken approximately 14 days after blasting. A slide has already blocked the tunnel. The photograph in figure 6 shows one of the slides at the Matre Hydro-electrical Power Plant. The area of the tunnel was approximately 24 m^2. It was filled with debris over a length of more than 90 m except for a 4 m^2 opening along the roof of the tunnel. The figure indicates the conditions downstream as viewed towards the outlet of

Figure 1. Fault zone with clay material at Randsfjorden

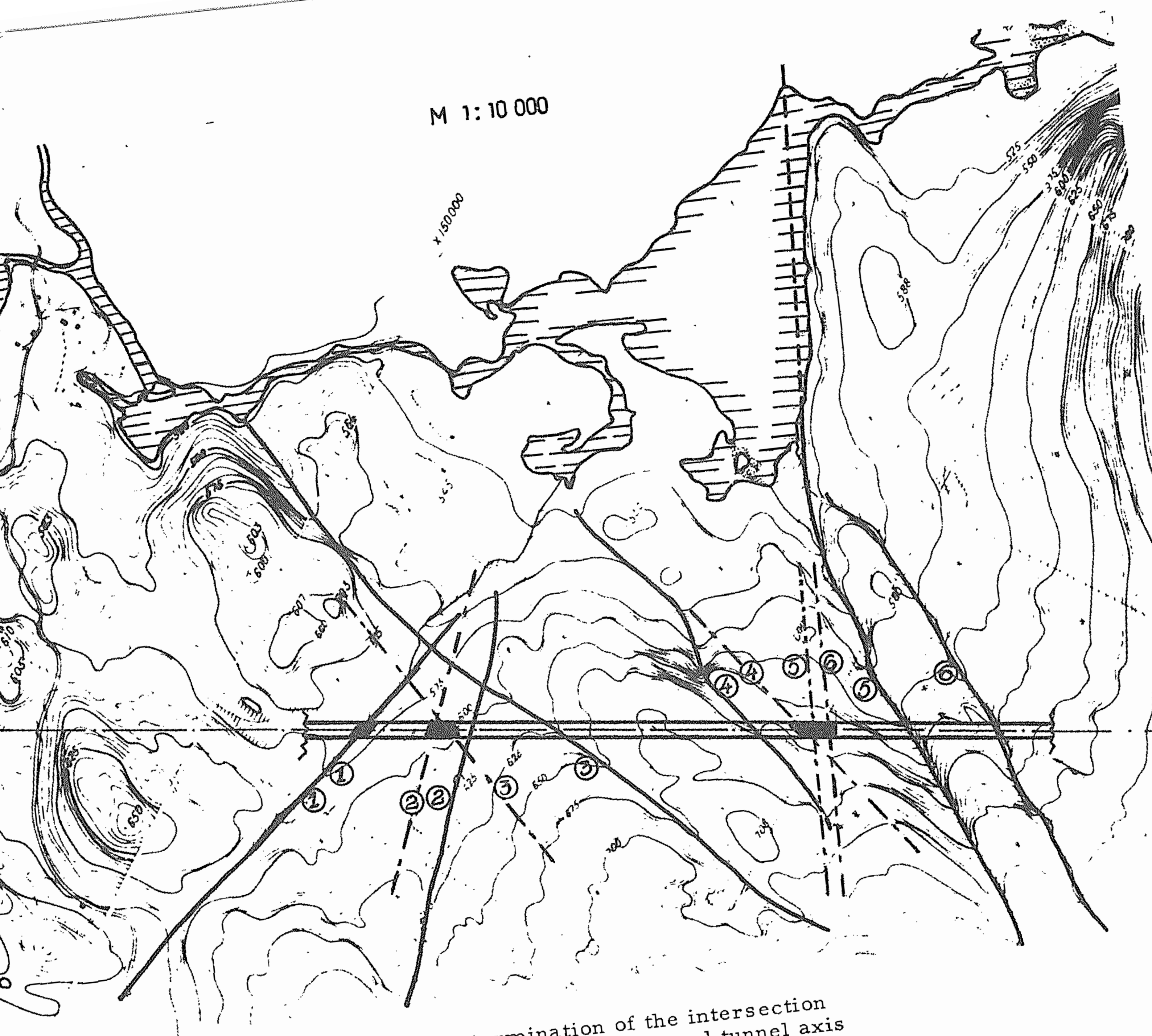

Figure 2. Determination of the intersection between clay zones and tunnel axis

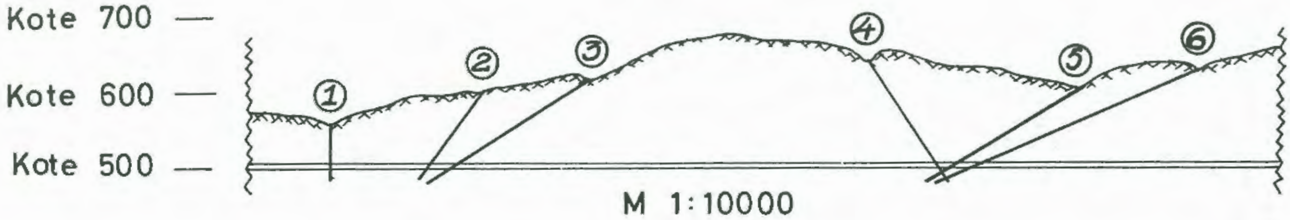

Figure 3. Profile along the tunnel axis. The location of the fracture zones has been determined as shown in figure 2

Figure 4. Clay zone where the feldspar has been altered into montmorillonite

the slide. Approximately 25 m^3 water per second has passed through the large blocks which blocked the tunnel below the failure zone for a distance of 4 m during a period of more than two years. The volume of the slide was approximately 2000 m^3. The width of the clay zone was less than 1/2 m, but apparently clay seams occurred in adjacent rock over a total length of approximately 4 m. The clay zone was supported with shotcrete. This is not an exceptional case. Many small and large slides have occurred in water tunnels supported by shotcrete. For instance, rock slides have occurred in a 60 m^2 tunnel in Sweden where the volume of the slide masses was approximately twice that mentioned above.

All clay zones are not equally dangerous and difficult. There are mainly four factors which affect the stability, viz. the orientation, thickness and structure of the clay zone, the degree of consolidation of the clay, the water conditions, and the mineralogical composition.

The thickness of the clay zone may vary from a few millimeters up to 50 meters. Also the character of the adjacent rock and the degree of fracturing may vary. This is often a result of the way the zones have been formed. We may distinguish between pure shear cracks and pure tension cracks and combinations of these two, in addition to the more complex fractures or crush zones. The different fracture mechanisms are illustrated in figure 7. The strike and dip of the clay zone with respect to the tunnel is important. This may, from the point of view of support, be favourable when the strike of the clay zones is nearly perpendicular to the tunnel wall, or unfavourable when the strike follows the tunnel axis.

In addition the degree of consolidation and the in-situ strength of clay material are important factors. These are normally governed by the rock-pressure perpendicular to the clay zone. But the clay might be very soft locally because of stressfree pockets, possibly constituted by cleft water pressure in vertical clay zones in area without any additional tectonic horizontal stresses in the direction perpendicular to the zones. The consolidation pressure is generally very high. In hydro-electric tunnels in Norway it often exceeds 1000 tons/m^2. The clays are very hard and apparently dry after blasting,

Figure 5. Example of a slide in a 6 m^2 tunnel through a clay zone.

The sliding started more than a week after the blasting. Skogn, Tröndelag

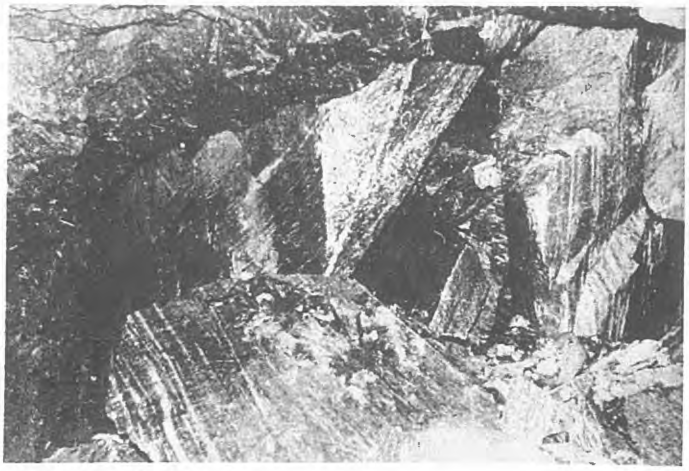

Figure 6. A "wall" of blocks in a water conveying tunnel caused by rock falls from the side of a clay zone with a width of about 0, 5 m.

The clay zone is perpendicular to the tunnel axis. Matre, Hordaland

and so compressed that the water content is below the hydroscopic by 90 percent relative humidity.

Furthermore, the local water conditions are important. Leakage occurs mainly through the clay-free joints at the side of the clay-filled zone. The relative humidity of the air in the tunnel is also very important because the water content in our swelling clays often is so low that the clay takes water from the air. If the tunnel later on is to be used for transport of water or is accessible to frost, the problems increase.

Concerning the mineralogical conditions it is far more important to consider the behaviour of the material on unloading and access of water than the mineral composition of the rock mass. Based on this, the clays may be divided into the following three groups:

1) Inactive clays which behave as Norwegian quaternary clays and do not change their consistency appreciably on unloading when they remain wet (for example, pure illite and kaolinite clays).

2) Clay containing material which loses its coherence along cracks when wet and unloaded. This desintegrates and falls apart without swelling. Such clays look like the quaternary clays which show a dry crust which is desiccated by cracks and which have a water content which approximately corresponds to the shrinkage limit (for example, pure clorite veins in gabbro).

3) There are finally those clays which on unloading by tunneling show an active swelling when they remain wet which is much larger than that caused by elastic deformations at unloading. These clays are called swelling clays. They contain smaller or larger amounts of montmorillonite or nontronite besides other clay minerals or unweathered minerals.

   Minerals which can be dissolved in circulating acid ground water should also be mentioned in this connection. These can cause leakage or lose their consistency a relatively short time after tunneling. To this group belong several kinds of massive and porous calcites.

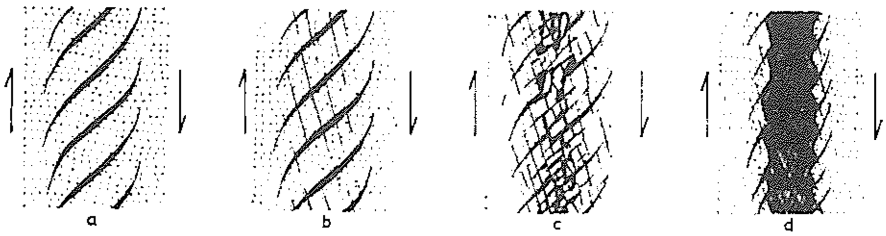

Figure 7. Development of a crush zone.
a = Feather joints.
b = Feather joints and shear joints.
c and d = Types of crush zones

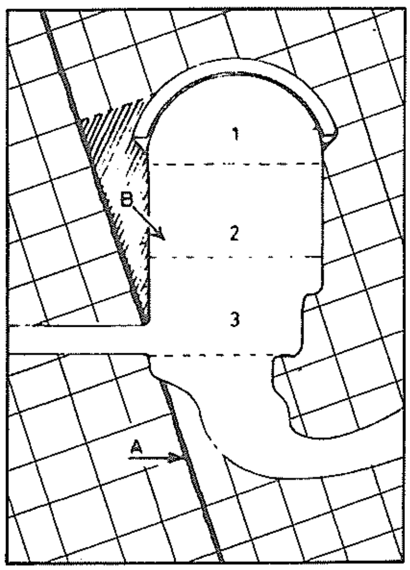

Figure 8. Section through a power plant showing a clay seam parallel with the axis of the hall (A). Part B slides out as soon as Section 3 is removed

The slide mechanisms are dependent on all the factors mentioned above and can therefore differ from place to place. All types of clays can cause slides if the consolidation is not sufficient to keep the rock mass in place in the roof or in the wall. All types of clays can be washed out or forced out by the pore water pressure if the leakage through the rock at the side of the clay zone or through the clay zone is sufficiently large.

If the leakage is small and the consolidation pressure is large, as is usually the case, only two of the above-mentioned clay types can cause large slides if the zone is reasonably perpendicular to the axis of the tunnel. These are those materials which lose their coherence when wet and those that swell. The zones of the first-mentioned type which are full of cracks, disintegrate gradually until an arching effect is established. The other clay zones are more or less forced out by their own swelling pressure as the water content increases and the shear strength of the clay is reduced. The moisture which normally is present in the air in the tunnel is sufficient to start the process except in very cold and dry periods close to the exhaust of the ventilation system. The relative humidity of the air in tunnels is as a rule very high, often close to 100 percent.

In these cases it is mainly the clay itself and the blocks embedded in the clay mass which fall out. This is also the most common type of large slides. Slides can, however, also be caused by unfavourable orientation of the clay zones in relationship to the roof and the walls. A clay zone may, for example, be parallel with the direction of the tunnel axis and intersect joints or another clay zone outside the tunnel profile. The rock mass located between the two zones may in this case fall out. In the walls it is mainly the steeply dipping clay zones which are dangerous. In the roof, zones which have moderate to small dips are the most dangerous. Also very small zones can have serious consequences (figure 8). These slides are not completely independent of the type of the clay material because the shear strength of many clay zones will change if water reaches them and the stresses in the rock will allow water to be taken up. The extent of these slides depends on the size of the tunnel profile and how the zones intersect the profile.

This is in contrast to the case where the debris mainly consists of the material from the clay zone and where the size of the slide is more or less independent of the size of the tunnel. For example, an opening of 1 m^2 in a steep clay zone in a water tunnel may cause a slide which involves several hundred cubic meters. In traffic tunnels the slide stops when the slope of the sliding mass corresponds to the angle of repose and the tunnel is filled up to slide out-let.

A large number of factors must be considered when the stability is evaluated: The size of the clay zone, the degree of fracturing of the adjacent rock, how the zone intersects the tunnel, if there are two or more closely spaced zones or zones which intersect, if free water is available and last, but not least, the clay material in the zones, its properties, degree of consolidation and compactness. A large number of these factors cannot be determined before the blasting of the tunnel or before the zones have been carefully explored by diamond drilling. As such work is very costly and time-consuming, pertinent borings are often only made to investigate cases where it is believed that troublesome conditions will arise.

It is often said that the clay minerals in clay zones are predetermined from the rock which the zones intersect. Therefore, it is often assumed that swelling clays cannot occur in certain rock materials. But all experience indicates that the expanding clay minerals can occur in crush zones in all type of rocks. The clay minerals can occur in two different ways: As hydrolysed minerals of the crushed material and of the feldspar in the adjacent rock, or in the same way as calcite in veins, zeolites etc., hydrothermally deposited from circulating water. The last type can occur everywhere and is independent of the petrography of the surrounding rock. Thick zones of montmorillonite have e.g. been found in pure marble and in pure quartzite. It must, however, also be mentioned that frequently mixtures of kaolinite and montmorillonite are often found in quartzite. Also in gabbro, mixtures of chlorite and montmorillonite are often found. Graphite is frequently the main constituent of clay zones in alum shales. It can therefore be said that there is often a relationship between the rock type and the composition of the clay material, but from a technical point of view such an information is of little help; it can be misleading also because a small amount of montmorillonite can change considerably the behaviour of a clay.

Often when swelling clays are encountered montmorillonite constitutes only a very minor component. The material may appear at first sight as a harmless pure chlorite or a bleached gneiss or granite. It is only on the basis of, e.g., DTA-analysis and swelling tests in an oedometer that it is possible to tell if the clay contains expanding mineral and is of a swelling type or not.

If the rock has a high content of feldspar it can also be affected by the circulating water in the failure zone to such an extent that the feldspar has irregularly changed into montmorillonite at the sides of the zone. This rock mass has kept its structure and can sometimes be so firm that the drill holes are visible after the blasting. The zones can often be detected because the feldspar has been bleached. It will not take many days before a swelling effect can be recognized if water comes in contact with the zone.

Because montmorillonite can be formed by hydrothermal processes as well as hydrolysis, montmorillonite is found in all types of joints and fracture zones in Norway, but fortunately not everywhere. It appears that all cracks through which water has circulated simultaneously contain clays having the same mineralogical composition. In one area clay zones with different orientations are found which have been formed under different periods of deformation and which have very different mineral compositions. There are also conditions that indicate a high age for the clays, for example late Caledonian or Permian. There are also indications that formations contaminated with montmorillonite occur locally with decreasing amount of the montmorillonite content and increasing amount of other minerals, radially from these areas. The contaminated area is often about 50 - 200 km^2.

There are no geological formations in Norway which are completely free from the problem of swelling clays, but the problem varies. We may distinguish between favourable regions and less favourable regions in connection with this problem.

The methods which are used to identify or classify swelling clays are DTA, X-ray refraction techniques and swelling tests. In addition, there are certain staining tests such as the malachite-green and the benzidine tests. We mainly use DTA, different types of staining tests, measurements of swelling pressures at constant volume and simple swelling tests with free swelling in water. However, the method we have developed for the measurement of swelling pressure at constant volume must be reevaluated because the preparation of the material is too complicated to be standardized.

We are today working with a method which is based on a type of consistency limits found by certain consolidation tests and the percentage of silt and clay size particles, approximately in the same way as for the activity ratio of clays.

We try by consolidation tests and DTA tests to develop new methods to evaluate the activity ratio and the maximum mobilized swelling pressure towards shotcrete linings. None of our tests today can be used to determine the swelling pressure which can be developed in a clay zone because of the time factor and because of the coarse material in the clay zone which makes it almost impossible to obtain undisturbed samples. However, it is generally not necessary to dimension an ordinary concrete lining on the maximum swelling pressure.

Problems of support caused by swelling clays

The question of supporting clay zones comes up during the design stage, during the time of excavation of the tunnel, and when deciding on the permanent support. At the design stage when the location of the tunnel is decided with respect to the clay zones, decisions are taken as to which zones should be avoided and which should be crossed by the tunnel. These decisions have a very large influence on the total costs of the project.

Another situation is encountered during the excavation of the tunnel when it is necessary to make temporary supports in order to protect the workers. If swelling clays are encountered, it is very important to cast the concrete as soon as possible before the swelling process makes for difficulties. Generally, it takes a few days before this occurs. An example may illustrate the situation.

At a hydroelectric plant in the southern part of Norway the contractor had on a Saturday morning drilled into a large zone with swelling clay. When the workers left the working face for the day, approximately one metre of a steep clay zone was exposed after blasting at the end of the shift. No work was done on Sunday, and on Monday morning scaling of the tunnel roof started. The scaling continued all day of Monday, and the conditions did not improve. The situation got worse during the night and on Tuesday it was necessary to stop the work because of the slide material which completely filled up the tunnel.

As it is dangerous, time-consuming and costly to pass such an active slide zone in a 50 m^2 tunnel, it was decided to go through the clay zone at another place in the vicinity. (Experience indicates that a local slide will not effect the consolidation of the clay zone to a distance of approximately twice the tunnel diameter to the side of the tunnel even after several years in tunnels carrying water.) This time the tunnel was not advanced into the clay zone until everything was prepared. The forms were placed immediately after blasting and the whole concrete lining was cast during a few days without difficulties. It has been possible in such cases to advance tunnels up to 20 m per week, but rates of 8 to 12 m per week are more common.

If the face and the roof of the tunnel are covered with shotcrete, a somewhat longer time is available for casting the concrete. Blocks in the areas where thin fissures with swelling clays occur can be bolted, but it is generally in jointed clay-free rock that this method is used. Temporary steel arch support and bracing is used less and less because of the problems connected with the construction of the permanent lining.

It is mainly the foreman or the resident engineer who decides what should be done to protect the workers at the tunnel face, and the settlement is often a quick matter of opinion, especially where clays occur in the rock. It should be possible to improve the evaluation with simple tests of the montmorillonite content of the clay zones and of the water conditions at the site. Such tests require, however, a field laboratory and, not least, time. The contractor and not the owner has the responsibility for the workers and therefore it is difficult for the owner to prescribe how the temporary support should be done. In Norway the contractor is always paid separately for the necessary temporary support and sometimes for difficult scaling too. Due to this, the extent of supports vary very much with the working crew.

During the last few years it has been tried to shotcrete the tunnel close up to the face in order to pass areas with difficult rock. A permanent concrete lining is constructed if the shotcrete fails. However, we have examples where shotcrete on swelling clay in tunnels carrying water has collapsed only after over one year. The rock at this place was impermeable and without any leakage. In other places where leakage exists or the adjacent rock is permeable, the shotcrete has failed within less than a week. This has caused doubts about shotcrete as a supporting method against swelling clays, especially in water tunnels. The method is, however, often used as a temporary support close up to the face. In these cases only the roof is shotcreted so that the conditions should not be completely hidden. The clay zones and water leakages are mapped, and the roof and the walls are stereo-photographed in colour before shotcreting. In this way it has been possible to design the permanent support at a later stage. In such cases it has often been necessary with a cast lining on shotcreted sections. If it is possible to cast a lining far out in the tunnel without interfering with the advance of the tunnel and to shotcrete close up to the face, the total cost of shotcreting and casting will be about the same as if casting is done directly and close up to the face, but time is saved.

The third situation occurs when the permanent support is designed. Then it is often the purpose of the project which is the governing factor. The requirements for traffic tunnels are often different from e.g., water tunnels. In water tunnels it is only necessary to prevent large rockfalls. The support of tunnels which convey water will be discussed somewhat more thoroughly in the following text because it is this type of tunnel which causes the most serious stability problems.

Often the central clay zones are more than 20 cm in width and the adjacent rock is full of thin clayfilled veins over a width of several metres. There is rarely any problem what to do with such dangerous zones of swelling clay. They generally show a lot of overbreak e.g. caused by the absorbtion of humidity from the air. It is simple to understand how it will develop if a lining is not cast.

Zones of this type also occur in leakage-free rock, and they may be exposed so late during the construction stage that no overbreaks or other danger signals occur before the tunnel is put in use. Furthermore the zones may have been shotcreted immediately after blasting and got no failure caused to the impermeable and leakage-free rock. In such cases the element of risk is easy to overlook. If, in addition, the feldspar in the adjacent rock is partly changed into montmorillonite, it is especially easy to overlook and underrate the element of risk since the dangerous part of the tunnel is very firm and has the same appearance as the surrounding unaltered rock. Slides in such zones are common in water tunnels in Norway, in addition to the cases where some few smaller, but closely spaced, zones occur.

In figure 9 is shown an example from the Hemsil Hydroelectric Plant. The parallel clay zones were about 10 cm in width. Separately they could be left without support in high quality rock or be covered by shotcrete. But as a unit with cracks in the interjacent rock it is dangerous. The part of the tunnel had been shotcreted.

Relatively simple supporting methods can be used for single small zones when the surrounding rock is of good quality and the spacing of the zones is larger than one to two tunnel diameters. The choice of method depends on the size and location of the clay zone. Sometimes anchor bolts are sufficient and the clay is allowed to flow out. This method is often chosen when the zone is almost perpendicular

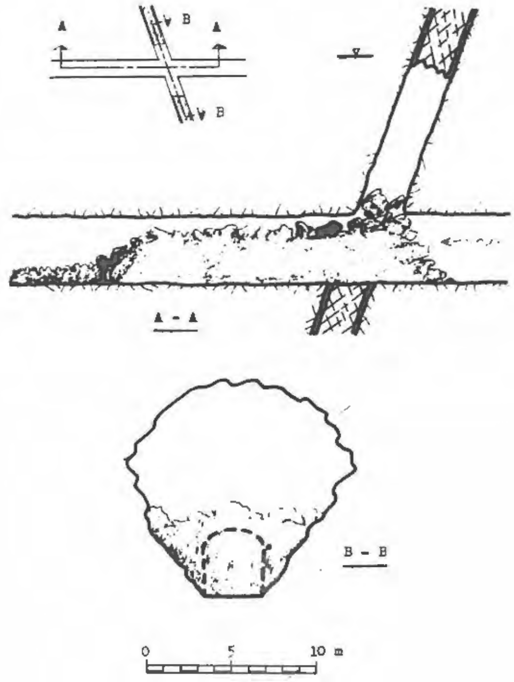

Figure 9. Sketch of a slide in a water conveying tunnel at Hemsil

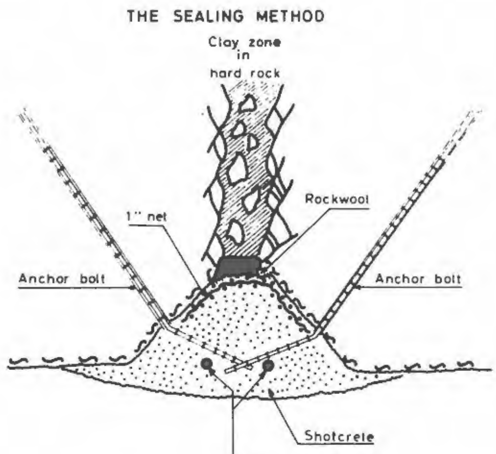

Figure 10. Sealing method

Figure 11. Heave of floor in a tunnel

to the axis of the tunnel and is clay-free in parts. If the situation is somewhat more complicated, the clay zone could be covered with reinforced shotcrete after it has partly been cleaned and after the clay has had time to swell. The extent of the support is often dependent on the requirements of the owner with respect to safety against rock falls.

If the zones are relatively large, up to a width of about 30 cm, it is possible in some cases to seal the zone with concrete. Strips of rockwool which have been covered by a fine wire mesh are then placed at the bottom of a blasted groove along the zone. The seal is anchored by bolts at both sides of the groove. The bolts are bent over the clay zone and covered by shotcrete (figure 10). It is necessary that the surrounding rock is of good quality, so that it is possible to use bolts. Furthermore that the clay content or the activity of the clay material in the zone is not too unfavourable.

The evaluation of the potential danger of a zone is to a large degree dependent on personal experience. There are so many factors which influence the stability as mentioned previously that it is very difficult to set up simple rules. When most clay zones are small and of a relatively harmless type and when the quality of the surrounding rock is high, it is essential to evaluate accurately the potential risks. The average costs for supporting clay zones in a 30 km long tunnel with a cross-sectional area of 30 m^2 is on the average over 2 mill. Norwegian crowns. If everything which may be called a clay zone is encased in concrete, the costs increase to more than the double. If a clay zone which has been overlooked fills up a water tunnel and has to be supported at a later stage, when the power plant is in use, the costs will often be five times larger than the costs for support during the construction stage. The costs for loss of production, which can be much larger, are not included here.

If the project, on the other hand, is easily accessible and is not used during parts of the year, smaller zones may be supported at a later stage. In this way the tunnel can be tested before the final supporting is done, but this is, as found from experience, seldom of current interest.

Clay zones which do not contain swelling clay can to a large extent be supported with shotcrete to prevent rock falls and erosion. Even if the clay zone is highly consolidated a certain maximum width of the clay zone and a certain maximum leakage can be allowed when using shotcrete.

One question that often arises is why shotcrete of good quality is not so good as an ordinary concrete lining with respect to swelling clays. One explanation is as follows: When shotcrete is sprayed on the rock immediately after the blasting, the clay has not had time to swell before the concrete hardens and often the shotcrete does not shrink as does ordinary concrete. This causes the full swelling pressure to act on the shotcrete and push holes in it. A great advantage with shotcrete is its ability to tie loose, clay-free blocks together with the solid rock in the tunnel, often in the form of an arch. If shotcrete is sprayed on clay it is the relatively thin irregular concrete shell alone which will form the arch and which must resist swelling pressures of up to a few hundred tons/m^2.

A usual concrete lining, on the other hand, will have poor contact with the rock and the full swelling pressure will not be mobilized. This is due to the fact that the forms are not completely filled, that concrete shrinks when it hardens, that pockets occur after rock falls and that the scaling of the walls has not been perfect. An expansion of less than 5 percent of the volume of montmorillonite to a thickness of three times the width of the zone is required in the roof to reduce the swelling pressures to a minimum. A much smaller expansion is required in the walls. Experience gained over 50 years indicates that clay located further back in the zone than three times the width of the zone will not change its state of stress because of silo action. The swelling pressures are here transferred to the rock on both sides of the zone. For example, a 3 cm clearance is required between the clay zone and the concrete lining at the roof in order to release the swelling pressures in a potentially dangerous clay zone with in average 10 percent of very active montmorillonite and a total width of 3 m. The contractor will satisfy this requirement automatically and with pleasure.

Shotcrete cannot resist pressures which can exceed 300 tons/m^2 for weeks or months after it has been sprayed on a clay zone without cracking or spalling. A non-reinforced concrete lining, 30 cm thick, cannot resist such high pressures, either. However, thousands of these unreinforced concrete linings are standing undamaged today because the casting has not been done with perfect contact to the rock.

A similar phenomenon occurs when clay freezes. This is not only the case for swelling clays. Damages on shotcrete caused by freezing, especially in connection with clay zones and scales have been a serious problem in some traffic tunnels.

Clay behind the concrete lining in tunnels carrying water can be washed out when the clay is relatively pure, especially when leakage occurs in the rock. The concrete should have good contact with the rock at both ends of the lining. Drain holes may be necessary in the side rock. In some cases the concrete lining must be designed to resist high water pressure, e.g. in pressure shafts without any steel tube and where zones of swelling clay are crossed by thick calcite veins in an area of non-calcareous rock. When the clay zone is wide, it is also necessary to prevent the clay from being squeezed out at the floor (figure 11). This can be done by excavating and replacing with coarse crushed stone and bracing with concrete beams between the walls.

The clay zones will normally contain so much coarse material that a natural filter develops which protects the zone against further erosion. A slide may develop if a large portion of the clay zone is washed away and the clay in the walls is sufficiently softened. If the stone content is small and the clay zone is very large, the tunnel floor must be protected by, e.g., an inverted arch with a porous base. Zones with swelling clays with a thickness exceeding 50 m in a 100 m^2 tunnel have been encountered where the normal lining cast during the advance of the tunnel must be considered only as a temporary support.

In many cases the problems can be very complex, especially when thick zones are encountered in large rooms or when the clay zones cut across large parts of the rock in the walls or in the roof in such rooms. Complex situations can also develop in combination with high rock pressures or with porous water bearing calcite zones in connection with swelling clays under acid conditions or in pressure shafts without steel tube, but with water pressures of several hundred metres. Such cases must be treated separately.

Another method that has been used in several cases against swelling clays is to inject behind a circular, very strong, reinforced concrete structure in order to distribute the swelling pressure uniformly. This is a very costly method, and is to be used only in special cases in Scandinavia where we not have squeezing rock with montmorillonite. We are lucky to have rock of high quality but in spite of this fact we should not underestimate or overlook the often very difficult problems which do occur.

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